GEI Consultants

Geotechnical
Environmental and
Water Resources
Engineering

April 15, 2005
Project 050030

Ms. JoAnn Fryer, P.E.
CLD Consulting Engineers, Inc.
Park Place Corporate Center
316 US Route 1, Suite D
York, Maine 03909

Subject: Geotechnical Design Study
Simpson Road Bridge Rehabilitation
Saco, Maine

Dear Ms. Fryer:

GEI Consultants, Inc. (GEI) has prepared this letter report to summarize the results of our geotechnical design study for the rehabilitation of the Simpson Road Bridge over Stackpole Creek in Saco, Maine. Our work on this project was authorized by the subconsultant agreement between CLD Consulting Engineers, Inc. (CLD) and GEI, dated December 30, 2004.

SITE AND PROJECT DESCRIPTION

The bridge is an earth-filled arch bridge constructed of stone masonry. The approach embankments on either side of the bridge are supported by stone masonry retaining walls. The overall structure, including approach embankments, is approximately 120 feet long, about 27 feet wide, and about 26 feet high at the location of the brook. The waterway opening for the brook is about 8 feet wide and 19 feet high, and is capped with a circular arch. The bridge is believed to be over 100 years old and is considered to be of historical significance.

The bridge is currently in poor condition. The stone abutment and retaining walls show signs of bulging in several locations and a few of the larger stones near the bases of the walls have been displaced. Staining and efflorescence were observed on the walls (as high as just below the springline of the arch) and appear to indicate the draining of water through the masonry after flood waters have receded or drainage of water infiltrating from above. A longitudinal crack in the abutment wall and arch has been noted by CLD. Longitudinal and transverse cracking of the roadway pavement behind the stone masonry retaining walls appears to indicate lateral movements of the walls. Sinkholes that have developed in the pavement in the vicinity of the guardrail posts appear to indicate infiltrating surface water has eroded backfill soils through the joints in the stone masonry. Due to the poor condition of the bridge, a steel frame bracing system has been installed in the waterway opening to support the abutment walls.

GEI visited the site with CLD personnel on June 11, 2002, and summarized our preliminary evaluation of the bridge in our letter dated June 20, 2002. In the letter, we discussed potential causes for the deterioration of the walls and the cracking of the pavements, and options for rehabilitation of the bridge. Options currently being considered for rehabilitating the bridge include:

- Excavation/Replacement of Wall Backfill: This option involves the removal of the backfill behind the abutment and retaining walls, casting concrete gravity or cantilever walls against the existing walls if necessary to increase their stability, and backfilling. The backfill would include measures to enhance drainage.

- Ground Anchor Reinforcement: This option involves the installation of grouted ground anchors to reinforce the walls and arch.

More detailed discussions of these and other options previously considered are summarized in our letter dated June 20, 2002.

SCOPE OF SERVICES

GEI has conducted a geotechnical study for the preliminary design phase of the project to further evaluate the rehabilitation options that are under consideration. GEI's scope of services for this preliminary design study has included the following:

- Planned and supervised a test boring program to obtain subsurface information concerning the bridge backfill and foundation conditions.

- Engaged a geophysical exploration consultant to conduct a ground-penetrating radar survey of the retaining walls that support the approach embankments.

- Performed four grain size analyses on samples of the backfill supported by the retaining and abutment walls.

- Evaluated the two rehabilitation options under consideration.

- Developed parameters for wall stability analyses to be conducted by CLD.

- Prepared this report.

SUBSURFACE EXPLORATIONS

Test Boring Program

GEI engaged Maine Test Borings, Inc. of Brewer, Maine to drill borings at the site. Four borings (B1 through B4) were drilled at the locations shown on Figure 2 on January 10 through

12, and February 16, 2005. GEI field engineers observed the borings. Driller's boring logs, with handwritten annotations by GEI, are provided in Appendix A.

The borings were advanced using a combination of hollow-stem auger and cased wash boring techniques to depths ranging from 18.3 for 26.0 feet below the pavement surface. A diamond spin casing shoe was often needed to advance the casing due to the presence of numerous cobbles and boulders in the fill. Split-spoon soil sampling with standard penetration tests (SPTs) was typically performed at intervals of 3 to 5.7 feet. Rock coring was used to drill through cobbles and boulders in borings B2 through B4 and to obtain bedrock samples in borings B1, B3, and B4. Boring B2 was abandoned at a depth of about 18.3 feet due to a broken diamond spin casing shoe, and did not encounter bedrock.

Upon completion of drilling, all of the borings were backfilled with soil cuttings and the pavement was repaired with cold patch.

Laboratory Testing

Laboratory grain size analyses were conducted on four samples of the backfill soils obtained from the borings. The results of the grain size analyses are provided in Appendix B.

Ground Penetrating Radar Survey

Kick Geoexplorations, Inc. (in cooperation with Radar Solutions, Inc.) conducted a ground penetrating radar survey to estimate the thickness of the masonry walls. The field survey was performed on February 1 and 2, 2005. The results of the survey are summarized in the report *Ground Penetrating Radar Survey, Simpson Road Bridge, Saco, Maine*, dated April 13, 2005, which is provided in Appendix C.

In general, the results of the radar survey indicate that the masonry walls have a trapezoidal shape, with thickness increasing with depth. The thickness of the taller portions of the walls (exposed heights on the order of 20 feet), at the level of the ground surface at the toe of the wall, ranged from about 8 to 10 feet, or 40 to 50% of the exposed height. The ratio of the wall thickness at the level of the ground surface (at the toe) to the exposed height typically increases with decreasing exposed heights, with thickness ranging from 58% to greater than 100% of the exposed wall height.

SUBSURFACE CONDITIONS

The subsurface conditions encountered in the borings are shown on the soil profile on Figure 3 and described below, from the ground surface down. Subsurface conditions are known only at the boring locations, and the conditions between borings may differ from those described in this report.

Asphalt

All four borings were drilled in the roadway and encountered asphalt ranging in thickness from about 2 to 9 inches.

Fill

Fill was encountered beneath the asphalt in all of the borings. The thickness of the fill at the boring locations ranged from 13.5 feet at B1 to 21 feet at B3. B2 was terminated in the fill at a depth of 18.3 feet, and the thickness of the fill at this location is not known.

The constituents in the fill are variable, but typically consist of silty sand and sandy silt with varying amounts of gravel. Numerous cobbles and boulders, and possibly stone masonry blocks, were encountered in all of the borings. Lower portions of the fill (below depths ranging from 6.1 to 10.9 feet) at B2 through B4 contained more cobbles and boulders than soil. Stone blocks encountered from a depth of 18.5 to 21 feet in B3 may be heel of the masonry wall.

SPT N-values in the fill were variable, due primarily to the numerous cobbles and boulders. Where cobbles and boulders were encountered, SPT N-values were usually high, ranging from 42 blows per foot to greater than 100 blows per foot (refusal), and sample recoveries were poor. In other areas, SPT N-values ranged from 18 to 25 blows per foot, indicating that the soil portion of the fill is medium dense. In some cases, SPT N-values in the lower range were obtained in conjunction with little or no sample recovery. This may indicate that the fill at these locations is very loose, allowing cobbles or boulders to be pushed through the fill ahead of the split-spoon sampler.

The results of grain size analyses of the backfill soils are provided in Appendix B.

Bedrock

Bedrock was encountered beneath the fill in B1, B3, and B4 at depths ranging from 13.5 (B1) to 21.0 feet (B3) below the pavement surface. B2 was terminated in the fill above the bedrock surface, and the depth to bedrock at this location is not known.

The bedrock surface ranges from about 1 (B3 and B4) to 4.5 feet (B1) below the ground surface at the toes of the adjacent sections of the masonry walls. The masonry blocks that were encountered at the bottom of the fill layer in B3 were placed directly on the bedrock surface. These observations indicate that the walls probably bear on the bedrock surface.

The bedrock in the core samples consists of a gray metamorphic rock. The rock is fine-grained and foliated. Joints in bedrock cores are spaced from about 1 to 17 inches, with dip angles of about 0°, 50°, and 70° to the horizontal. The rock is slightly to moderately weathered.

Bedrock outcrops in the vicinity of the bridge were not visible during the drilling program due to snow cover. However, during the site visit for the preliminary evaluation, outcrops were observed in the bottom of the creek and along the west bank of the creek upstream and downstream of the bridge. Outcrops were not observed along the east bank of the creek, but this area was obscured by vegetation.

Groundwater

Observation wells were not installed in the borings for the measurement of groundwater levels. Also, the borings were drilled using cased wash boring drilling techniques, which use water to flush cutting from the borehole, making it difficult to estimate groundwater levels based on soil moisture content and water levels in the borings upon completion. However, we expect that groundwater levels in the wall and abutment backfill are usually at or near the ground surface (or the water level in the brook) at the toes of the walls. During periods of heavy precipitation or snow melt, the groundwater level in the wall and abutment backfill probably rises due to infiltration through the pavement and, when the water level in the brook rises, through the faces of the masonry walls. As the water level in the brook recedes, the groundwater in the backfill slowly drains through the joints in the masonry walls. We expect that drainage through the face of the wall occurs more slowly than the rate at which the brook recedes, causing differential water pressure to act on the backs of the walls.

EVALUATION

As indicated in our letter of June 20, 2004, it appears that the deteriorated condition of the abutment and retaining walls is related to the following:

1) Earth and Water Pressures: The lateral displacements and bulging observed are typical symptoms of inadequate support of earth and water pressures. Water may be entering the backfill behind the walls as groundwater from the upslope areas to the east and west, as infiltration from above through cracks in the roadway, and directly through the walls during high flow events in the creek. Water infiltrating through the faces of the walls during high flow events may drain back more slowly than the flood waters recede, thereby temporarily increasing the hydrostatic water pressure against the backs of the walls.

   We recommend that stability analyses be conducted to determine the degree to which the earth and hydrostatic water pressures behind the walls are contributing to the deterioration. Parameters for use in analyzing the stability of the walls are provided below:

   a) Wall Geometry: We recommend that wall geometry for use in the stability analyses be based on the results of the radar survey provided in Appendix C. For sections of the walls with exposed heights greater than 10 feet, the wall geometry can be approximated assuming the wall thickness is 4.5 feet at the top, and increases by 0.25 feet per vertical foot below the top of the wall. If sections of the walls with exposed heights of 10 feet or less are analyzed, the wall geometry should be selected from the results of the radar survey. The wall sections should be assumed to be embedded about 1 foot below the ground surface at the toe (i.e., the total wall height should be assumed to be 1 foot greater than the exposed height). However, passive pressure at the toes of the walls should be neglected. The unit weight of the masonry should be assumed to be 140 pounds per cubic foot (pcf).

b) Base Friction: For the sliding analysis, we recommend that the friction along the base of the wall be estimated assuming a friction coefficient (tan δ, where δ = interface friction angle) of 0.55. A friction coefficient of 0.7 should be used for estimating sliding resistance through intermediate levels of the walls for the evaluation of the internal stability. The larger coefficient considers interlocking between courses of the stone masonry.

c) Earth and Hydrostatic Water Pressures: Our recommended earth, surcharge, and hydrostatic water pressures for use in the stability analyses are provided on Figures 4 and 5.

i) For the normal loading condition, the earth pressure behind the walls should be estimated assuming the groundwater level in the backfill is low enough that no differential water pressure acts on the wall. For this case, the active pressure should be used in conjunction with the traffic surcharge pressure, as shown on Figure 4. The active earth pressure was estimated assuming an internal friction angle of 30° for the backfill, an interface friction angle between the wall and backfill of 15°, and a soil total unit weight of 120 pcf. The traffic surcharge pressure was estimated assuming an equivalent 2-foot-thick layer of soil.

ii) An extreme loading condition in which a portion of the backfill is assumed to be saturated should also be analyzed. Partial saturation of the backfill would occur during storm events when the surface water in Stackpole Creek temporarily rises against the walls of the bridge, and then recedes more quickly than the water in the backfill can drain through the masonry, causing a rapid drawdown condition. The differential water pressure acting on the walls during the rapid drawdown condition depends on the rate at which the water infiltrates and saturates the backfill, and then drains from the backfill as the surface water level recedes. Estimation of the differential water pressure would require a transient flow analysis that is beyond the scope of this study. In lieu of a transient flow analysis, we recommend that the differential water pressure be estimated assuming that the groundwater level in the backfill remains at the water level associated with the 100-year storm event (elevation 68.2 feet, project datum), while the surface water in the creek quickly recedes. Earth and water pressures for this condition are provided on Figure 5. Uplift water pressure was estimated assuming the pressure decreases linearly from the heel to the toe of the structure.

iii) The stability analyses should also consider earthquake loading. According to the AASHTO Standard Specifications for Highway Bridges, 2002, (AASHTO Specifications), the design horizontal earthquake acceleration for the site is 0.095g, which represents an earthquake with a 10% probability of being exceeded in 50 years. Based on the AASHTO Specifications, the site meets the criteria for Seismic Performance Category (SPC) B. Since the bridge is a single span structure, the AASHTO Specifications do not require the detailed seismic analysis and design requirements for SPC B bridges. However, Section 5 of the AASHTO Specifications indicates that retaining walls should be designed for seismic earth pressures in accordance with Section 6 of Division I-A, which provides seismic design requirements for SPC B bridges. As such, we recommend that the stability analyses of the masonry walls include a horizontal seismic earth pressure increment as shown on Figure 4. This incremental pressure is to be applied in addition to the earth pressures described above for the normal loading condition.

iv) Factors of safety for the static loading conditions should be at least 1.5 for both sliding and overturning. For the seismic loading condition, the minimum factor of safety can be reduced to 1.125.

2) Freeze-Thaw Pressures: The displacement of stones from the faces of the walls is caused, at least in part, by freeze-thaw of water between stones. The displacement of stones from the faces of the abutment walls may also be caused by water forces imposed during periods of high flows.

3) Frost Heaving: The lateral displacement and bulging in the upper portions of the walls is caused, at least in part, by frost heaving of the backfill soil within the depth of frost penetration from the pavement surface. The backfill soils consist primarily of silty sand and sandy silt, which are considered highly susceptible to frost heaving. Also, infiltration through the cracks in the pavement is expected to provide sufficient water for the formation of the ice lenses that cause frost heaving. Frost penetration is expected to extend to a depth of about 4 to 5 feet below the pavement surface. Ice lenses and frost heaves probably do not form in the backfill below the upper 4 to 5 feet of the walls, because the masonry walls appear to be thick enough to prevent frost from penetrating to the backfill soils. However, pressures generated by frost heaving in the upper 4 to 5 feet of the walls could cause movements in lower portions of the walls.

   The pressures that can develop in the upper 4 to 5 feet of the walls due to frost heaving are difficult to quantify. We have not included these pressures in the loads provided for the evaluation of wall stability. However, we recommend that both options for bridge rehabilitation that are currently under consideration include measures to increase drainage and reduce frost pressures in the upper 4 to 5 feet of the walls.

BRIDGE REHABILITATION RECOMMENDATIONS

Additional evaluation and design recommendations for the bridge rehabilitation options currently under consideration are provided below.

<u>Excavation/Replacement of Wall Backfill</u>

Based on the results of the recent field studies, complete excavation and replacement of the backfill and construction of concrete backing walls may not be feasible. The results of the borings indicate that the backfill contains numerous cobbles and boulders, particularly below depths ranging from about 6 to 11 feet from the pavement surface. Also, the GPR survey indicates that higher portions of the walls have base thicknesses on the order of 10 feet at the level of ground surface at the toes of the walls. Since the bridge is approximately 27-feet-wide, the distances between the heels of the adjacent wall sections with heights of 20 feet or more are expected to be 7 feet or less. Due to the numerous boulders in the lower portions of the backfill, and the limited distance between the lower portions of adjacent sections of the walls, complete excavation and replacement of the backfill would require very careful control to avoid damaging the walls. It may be difficult to distinguish between boulders and masonry blocks in the backfill and the masonry stones that form the backs of the walls. Also, removal of all of the backfill would relieve horizontal pressures on the wall, which could cause the masonry to shift and possibly become unstable.

We recommend that this option be modified to include partial excavation and replacement of the backfill in conjunction with drainage improvements to reduce water infiltration into the backfill. The partial excavation/replacement option would include the following:

- The existing backfill should be removed to a depth of 5 feet. The subgrade of the excavation should be crowned to promote drainage towards the sides of the bridge, and the existing fill should be proof-rolled by several passes of a vibratory plate compactor. Large vibratory roller compactors should not be used.

- A minimum 2-foot-thick by 2-foot-wide zone of crushed stone (AASHTO #67 stone) should be placed on the excavation subgrade along the stone masonry walls to create drains. Weep holes should be drilled through the masonry walls near the level of the bottoms of the drains at longitudinal spacings of 3 feet. A nonwoven geotextile with a minimum weight of 8 ounces per square yard should be placed over the entire excavation subgrade and crushed stone, and continued vertically up against the inside faces of the stone masonry walls.

- The excavation should be backfilled with free-draining sand or sand with gravel containing less than 5% fines (soils passing the No. 200 sieve) up to the level of the pavement base soil. The new fill should be placed in maximum 9-inch-thick loose lifts and be compacted to at least 92% of maximum dry density as determined in accordance with AASHTO T180 using a vibratory plate compactor.

- A minimum 12-inch-thick layer of base soils should be placed over the backfill. The base soils should meet the requirements for crushed gravel aggregate per Section 703.06(a), Type A or B, of the Maine Department of Transportation Standard Specifications for Highways and Bridges (latest revision). The base soil should be compacted to at least 95% of maximum dry density using a vibratory plate compactor.

Removal of the upper 5 feet of existing backfill will require careful excavation. We expect that hand excavation will be needed along the backs of the walls to avoid dislodging stones from the masonry. Also, excavation of backfill from above the arch will reduce vertical stress without a corresponding reduction in horizontal stress on the abutments, which could cause the stones to shift. The steel beams placed in the waterway opening to support the arch and abutments will reduce horizontal movements of the abutments provided the beams are properly sized and fitted snugly against the masonry with wedges. Excavation and replacement of the backfill above the arches could be performed in narrow strips to further reduce the risk of unloading the top of the arch.

Partial excavation/replacement of the existing backfill would provide the following benefits:

- The existing frost susceptible backfill soil would be removed from within the depth of frost penetration (4 to 5 feet), reducing the effects of frost heaving on the walls and the pavement.

- The geotextile placed between the new fill and the masonry walls will prevent the new fill from eroding through the joints in the masonry, and reduce the potential for the formation of voids and sinkholes.

- Drainage of infiltration will be improved in the upper portions of the backfill. The reduction of infiltration into the underlying backfill will reduce the loss of backfill through the joints in the masonry and the formation of sinkholes. Hydrostatic water pressures caused by water infiltrating from the pavement will also be reduced. However, water will still be able to infiltrate into the face of the masonry during periods of high flow in the creek. When the creek level recedes, the water in the backfill will increase hydrostatic differential pressures on the walls. As the water drains from the backfill, erosion of backfill soils could continue, causing the formation of voids in the backfill that could, over time, be expressed as sinkholes. The geotextile placed between the existing backfill and the new fill will slow but not prevent the formation of sinkholes.

Partial excavation/replacement would also provide the opportunity to reinforce the masonry arch. CLD had observed a crack in the masonry abutment walls and arch perpendicular to the axis of the waterway opening. After removal of the backfill, a two-way reinforced concrete mat could be cast over the top of the arch to limit further movements. We expect that the mat could be pinned to the arch using dowels drilled into the stone masonry.

If the stability analyses indicate that additional measures are needed to improve wall stability, we recommend that the partial excavation/replacement of the backfill be done in conjunction with the stability improvement measures.

Ground Anchor Reinforcement

If the results of the stability analyses indicate that measures are needed to improve the stability of the walls, ground anchors could be considered. Ground anchors are installed by grouting steel reinforcing anchors into pre-drilled holes. Some considerations for the design of ground anchors follow:

- Anchors could be used to provide support for the arch parallel to the axis of the waterway opening (perpendicular to the crack observed in the arch and abutment walls). As previously indicated, it may be possible as an alternative to reinforce the arch by casting a two-way reinforced concrete mat over the top during partial excavation/replacement of the existing backfill.

- Anchors could be used to provide additional support for the walls. The anchors could be drilled through one wall and the backfill and into the opposite wall, to tie the two walls together. The drill could be stopped about 1 to 2 feet short of penetrating the far wall to limit disturbance to the appearance of the wall. The hole in the wall that is completely penetrated by the drill could be patched by grouting or epoxying in a core sample of the stone drilled to start the hole. As an alternative to tying opposite walls together with anchors, the anchors installed in the lower portions of the walls could be angled downward and installed into the bedrock that underlies the walls.

- The spacings between anchors should not exceed the thickness of the wall at the anchor locations. The anchors should be designed for a maximum load not exceeding the anchorage capacity of the anchor in the masonry wall.

- The anchors should be suitably protected from corrosion. The various anchor manufacturers have different proprietary corrosion protection systems. The anchors should meet or exceed the "double-corrosion protection" standards for anchors manufactured by Dywidag.

- The anchors should be designed in accordance with *Recommendations for Prestressed Rock and Soil Anchors*, published by the Post-Tensioning Institute, latest revision.

- We recommend the anchors be post-tensioned to about 10% to 20% of the design load to seat the anchors and reduce the amount of deflection needed to provide support for the walls. The anchors should not be stressed above this level to avoid damaging the stone masonry. During stressing, the stone masonry wall in the vicinity of the head of the anchor should be carefully observed for signs of deflection. Stressing should be discontinued if deflections become excessive.

Another method of improving wall stability would be to cast a concrete berm along the toes of the walls. The berm would be cast on sound bedrock and be secured with passive steel dowels grouted or epoxied into the bedrock and the masonry walls. The top of the concrete berm should be at least one foot above the normal water level in the creek. The concrete berm would increase sliding resistance along the base of the wall and may reduce the number of ground anchors needed to improve wall stability. The berm would also fill in gaps between the bedrock and masonry and help protect the stone masonry from debris carried by the creek.

LIMITATIONS

Our preliminary recommendations are based on the project information provided to us at the time of this report and may require modification if there are any changes in the nature or design of the project.

The recommendations in this report are based in part on the data obtained from the subsurface explorations. The nature and extent of variations between explorations may not become evident until construction. If variations from the anticipated conditions are encountered, it may be necessary to revise the recommendations in this report. Therefore, we recommend that GEI be engaged to make site visits during construction to:

1. Check that the subsurface conditions exposed during construction are in general conformance with our design assumptions.

2. Ascertain that, in general, the work is being performed in compliance with the contract documents and our recommendations.

Our professional services for this project have been performed in accordance with generally accepted engineering practices; no other warranty, express or implied, is made.

Please call if you have any questions.

Sincerely,

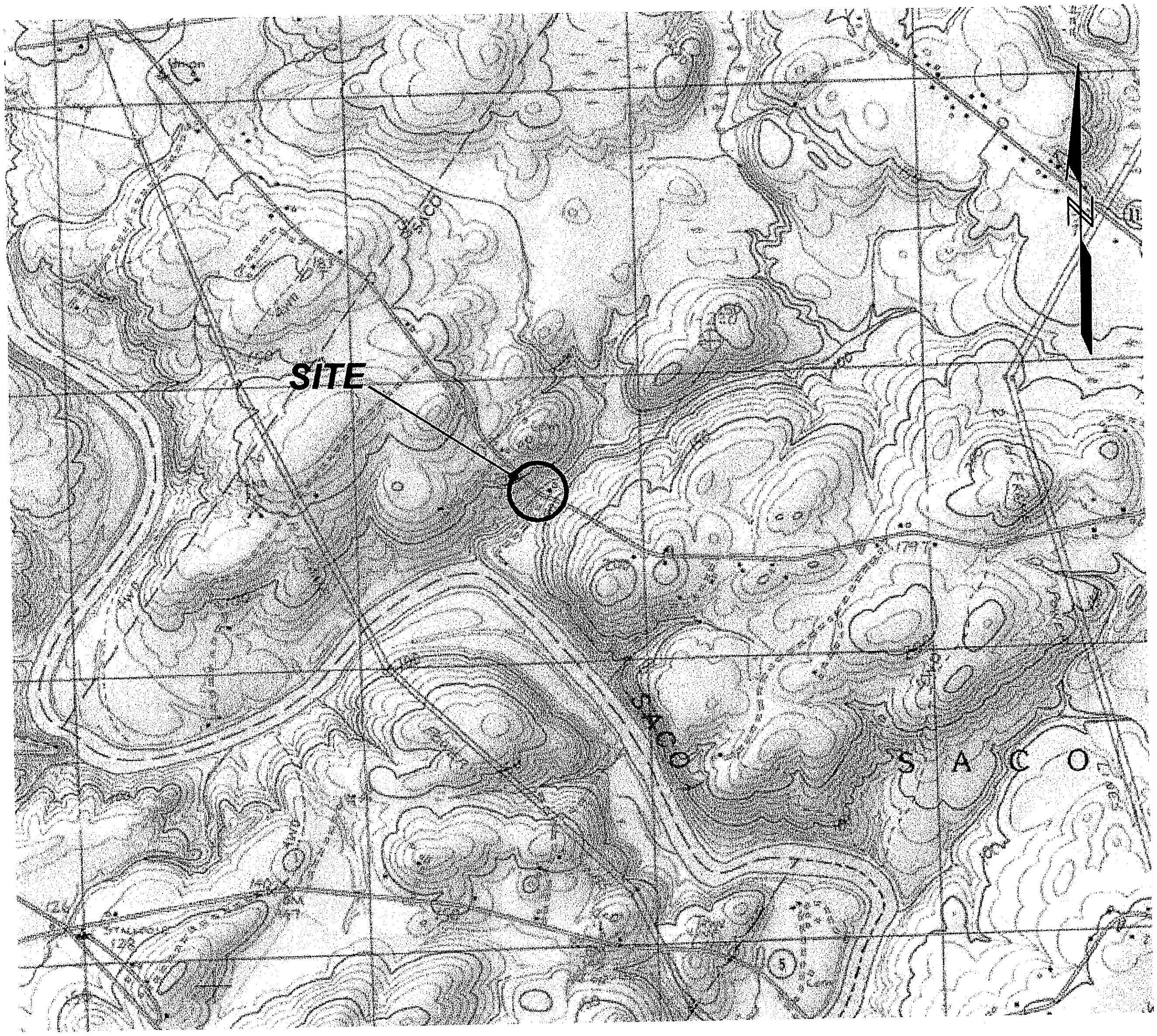
GEI Consultants, Inc.

Craig F. Ward

Craig F. Ward, P.E.
Manager – New Hampshire Division

CFW:

Figures
Appendices



1. DRAWING REPRODUCED FROM UNITED STATES GEOLOGICAL SURVEY (USGS) 7.5 MINUTE SERIES PROVIDED BY TOPO! NATIONAL GEOGRAPHIC HOLDINGS, INC. 2000.
2. CONTOUR INTERVAL IS 10 FEET (NATIONAL GEODETIC VERTICAL DATUM, NGVD, OF 1929)

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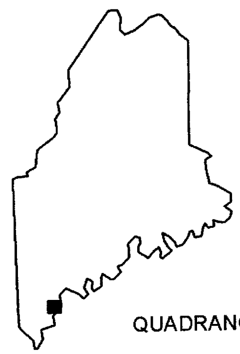
SCALE, MILES: 0 — 0.5 — 1.0



QUADRANGLE LOCATION

Simpson Road Bridge Rehabilitation Saco, Maine	GEI Consultants	SITE LOCATION MAP
CLD Consulting Engineers, Inc. York, Maine	Project 050030	April 2005 Fig. 1

050030 Figures.dwg CSM/als 4/12/05

NOTES

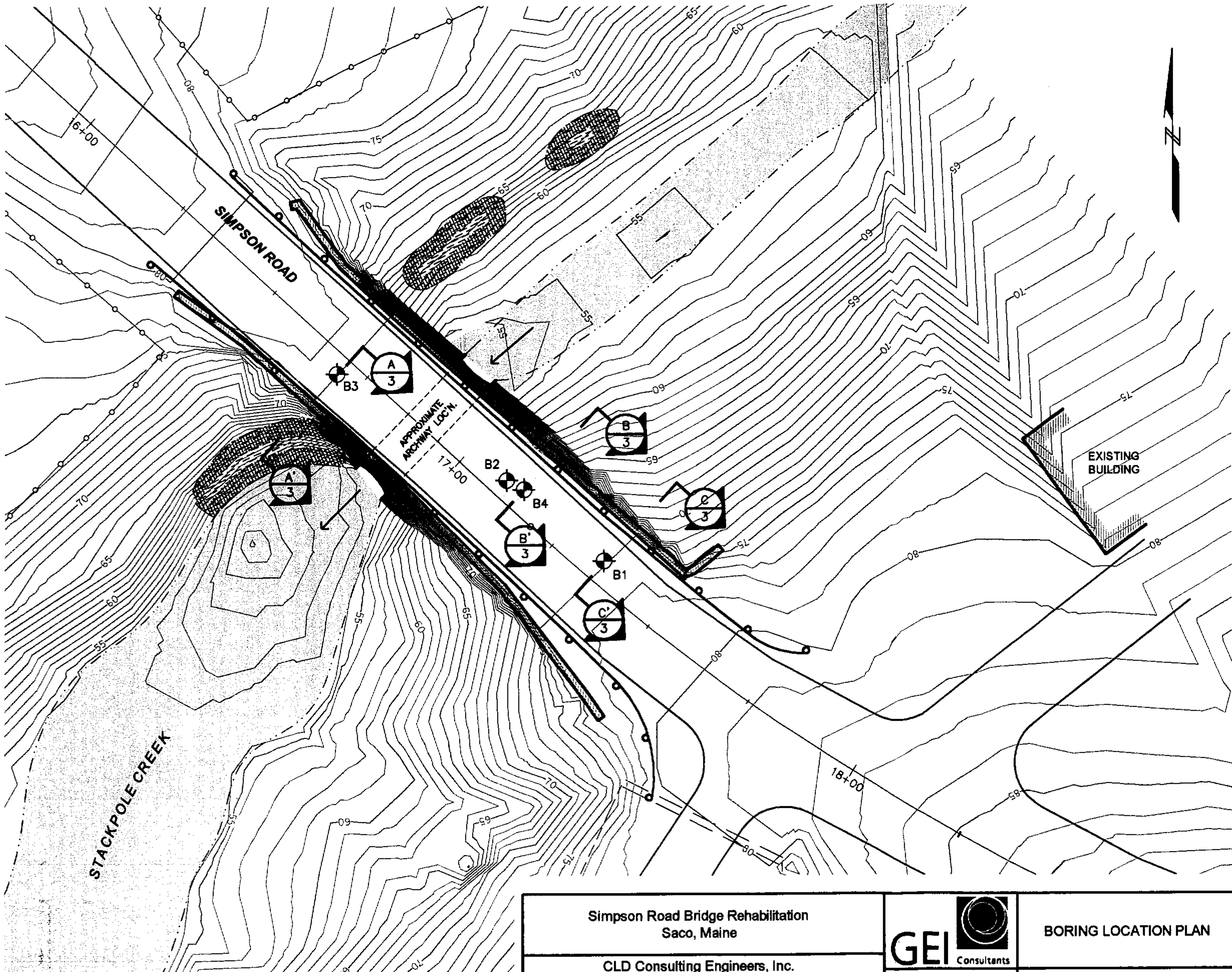
1. FIGURE REPRODUCED FROM DRAWING PROVIDED BY CLD CONSULTING ENGINEERS, INC., JANUARY 14 2005.
2. BORINGS DRILLED BY MAINE TEST BORING, INC., ORRINGTON, ME, JANUARY 10 TO JANUARY 12 AND FEBURARY 16, 2005.

LEGEND

B1	GEI BORING
	EXISTING RETAINING WALL
	EXISTING METAL BEAM RAIL
	EXISTING FENCE
70	GROUND SURFACE CONTOUR LINES
17+00	ROADWAY BASELINE AND STATION
	APPROXIMATE WATER'S EDGE
	POSSIBLE BEDROCK OUTCROP
A 2	SECTION LINE

APPROXIMATE SCALE, FEET

0 20 40



Simpson Road Bridge Rehabilitation Saco, Maine	GEI Consultants	BORING LOCATION PLAN	
CLD Consulting Engineers, Inc. York, Maine	Project 050030	April 2005	Fig. 2

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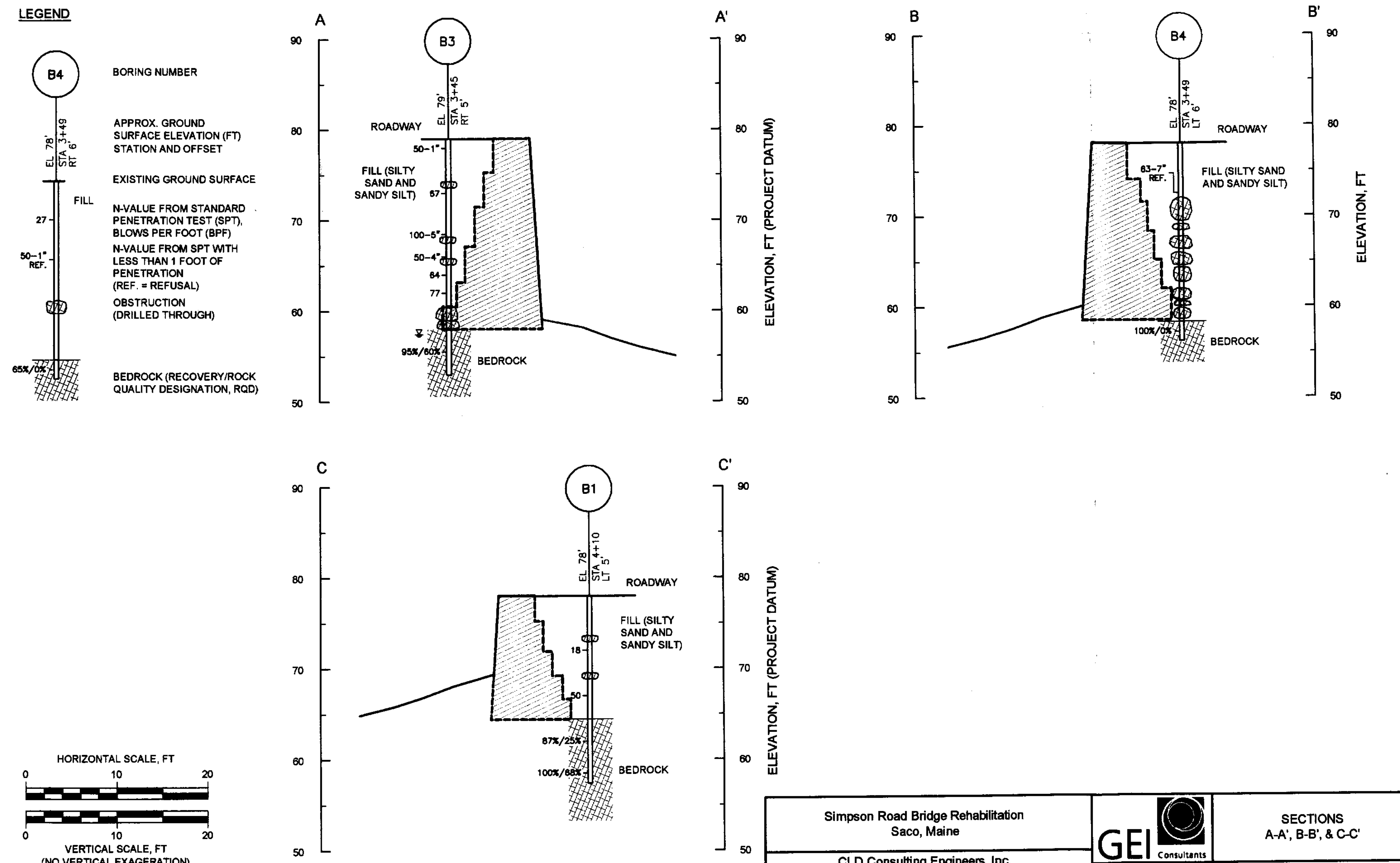


Simpson Road Bridge Rehabilitation
Saco, Maine

CLD Consulting Engineers, Inc.
York, Maine

GEI Consultants

Project 050030

SECTIONS
A-A', B-B', & C-C'

April 2005

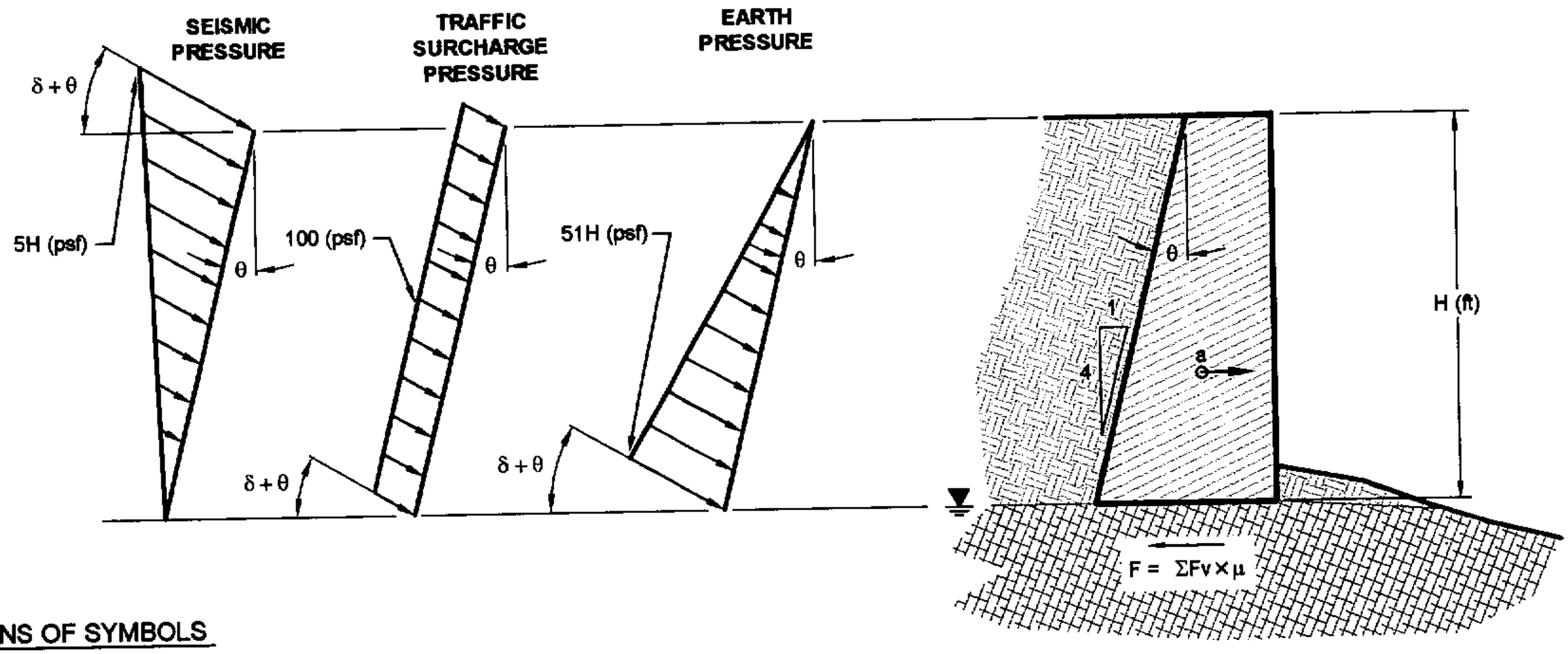
Fig. 3

050030 Figures.dwg CSM/als 4/12/05

NOTES

1. SEE FIGURE 5 FOR THE RAPID DRAWDOWN CONDITION.

DRAWING NOT TO SCALE



DEFINITIONS OF SYMBOLS

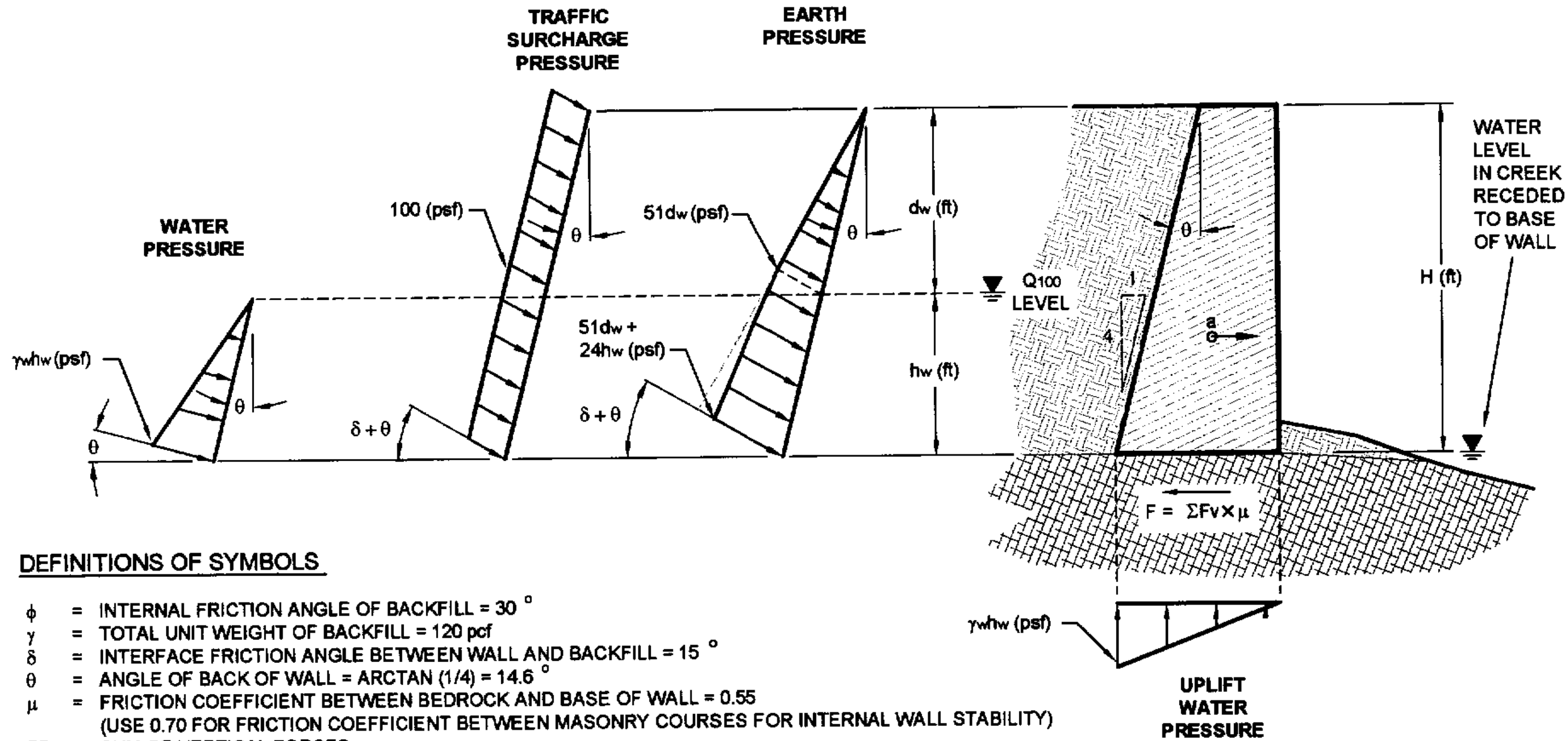
ϕ = INTERNAL FRICTION ANGLE OF BACKFILL = 30 °
γ = TOTAL UNIT WEIGHT OF BACKFILL = 120 pcf
δ = INTERFACE FRICTION ANGLE BETWEEN WALL AND BACKFILL = 15 °
θ = ANGLE OF BACK OF WALL = ARCTAN (1/4) = 14.6 °
μ = FRICTION COEFFICIENT BETWEEN BEDROCK AND BASE OF WALL = 0.55
(USE 0.70 FOR FRICTION COEFFICIENT BETWEEN MASONRY COURSES FOR INTERNAL WALL STABILITY)
ΣFv = SUM OF VERTICAL FORCES
F = BASE FRICTION
γ_{wall} = TOTAL UNIT WEIGHT OF MASONRY = 140 pcf
a = HORIZONTAL SEISMIC ACCELERATION = 0.095g

Simpson Road Bridge Rehabilitation Saco, Maine	GEI Consultants	EARTH AND WATER PRESSURES FOR NORMAL AND EARTHQUAKE LOADING CONDITIONS
CLD Consulting Engineers, Inc. York, Maine	Project 050030	April 2005 Fig. 4

050030 Figures.dwg CSM/ois 4/12/05

NOTES

1. SEE FIGURE 4 FOR THE NORMAL AND EARTHQUAKE LOADING CONDITIONS.



DEFINITIONS OF SYMBOLS

ϕ = INTERNAL FRICTION ANGLE OF BACKFILL = 30 °
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(USE 0.70 FOR FRICTION COEFFICIENT BETWEEN MASONRY COURSES FOR INTERNAL WALL STABILITY)
ΣF_v = SUM OF VERTICAL FORCES
F = BASE FRICTION
γ_{wall} = TOTAL UNIT WEIGHT OF MASONRY = 140 pcf
a = HORIZONTAL SEISMIC ACCELERATION = 0.095g
Q_{100} LEVEL = WATER LEVEL DURING 100 YEAR STORM = ELEVATION 68.2 FT (PROJECT DATUM)

Simpson Road Bridge Rehabilitation Saco, Maine	GEI Consultants	EARTH AND SURCHARGE PRESSURES FOR RAPID DRAWDOWN CONDITION
CLD Consulting Engineers, Inc. York, Maine	Project 050030	April 2005 Fig. 5

050030 Figures.dwg CSM/ols 4/12/05

MAINE TEST BORINGS, INC. BREWER, MAINE 04412	CLIENT GEI Consultants, Inc.	SHEET 1 OF 1 HOLE NO. B1
DRILLER Brad Enos	PROJECT NAME Simpson Road Bridge	LINE & STATION
M. . JOB NUMBER 05-008A	LOCATION Saco, ME	OFFSET

GROUND WATER OBSERVATIONS

	CASING	SAMPLER	CORE BARREL	DATE	DATE
TYPE	HW	SS	NQ2	02/16/05 Start	02/16/05 Finish
SIZE I.D.	4"	1 3/8"	2"	SURFACE ELEVATION	
HAMMER WT.	300#	140#			
HAMMER FALL	16"	30"			

CASING BLOWS PER FOOT	SAMPLE NO.	O.D.	PEN.	REC.	DEPTH @ BOT.	BLOWS PER 6" ON SAMPLER 0-6	6-12	12-18	18-24	VANE READING	DEPTH	STRATUM DESCRIPTION
Auger											~~0.5~~ 0.2'	Pavement
											3.5'	SILTY SAND (SM) - MOSTLY SAND, ABOUT 45 % SLIGHTLY PLASTIC FINES, OCC. GRAVEL FILL
"	1D	2"	24"	21"	7.0	8	9	9	9			BROWN SANDY SILT & Brown Silty Gravelly Sand w/Cobbles
54												
	2D	2"	24"	9"	12.0	57	21	29	23			
60"											13.5'	FILL
	1R	3"	4.7'	4.1'	18.3	87.2%						BEDROCK - METAMORPHIC, FINE-GRAINED, FOLIATED, JOINTS DIPPING 70° AT 1"-17" SPACINGS WITH ORANGE STAINING, SLIGHTLY TO ~~Rock~~ MODERATELY WEATHERED, DARK GRAY. (SCHIST) RQD = 25% IN 1R
	2R	3"	2.3'	2.4'	20.6	104%					20.6'	RQD = 68% IN 2R
												Bottom of Boring @ 20.6' Caved @ 19.6' Water @ 11.0'

SAMPLES	SOIL CLASSIFIED BY:	REMARKS:
D SPLIT SPOON R = ROCK CORE C = 2" SHELBY TUBE S = 3" SHELBY TUBE V = VANE	XX DRILLER-VISUALLY SOIL TECHNICIAN-VISUALLY LABORATORY TESTS	Washed ahead from 13.5' to 13.6' 60" for 0.5'

HOLE NO. B1

MAINE TEST BORINGS, INC. BREWER, MAINE 04412	CLIENT: GEI Consultants, Inc.	SHEET 1 OF 1 — HOLE NO. B-2
DRILLER: Ron Idano	PROJECT NAME: Simpson Rd. Bridge	LINE & STATION
M.T.B. JOB NUMBER: 05-008	LOCATION: Saco, ME	OFFSET

GROUND WATER OBSERVATIONS

—

	CASING	SAMPLER	CORE BARREL	DATE	DATE
TYPE	NW	SS	NQ2	01/11/05 Start	01/12/05 Finish
SIZE I.D.	3"	1 3/8"	2"	SURFACE ELEVATION	
HAMMER WT.	300#	140#			
HAMMER FALL	16"	30"			

CASING BLOWS PER FOOT	SAMPLE NO.	O.D.	PEN.	REC.	DEPTH @ BOT.	BLOWS PER 6" ON SAMPLER 0-6	6-12	12-18	18-24	VANE READING	DEPTH	STRATUM DESCRIPTION
Auger											0.6	Pavement
	1D	2"	24"	13"	4.0	67	15	10	12			BROWN SILTY SAND & Brown Sandy Silt w/Gravel
	2D	2"	24"	8"	7.0	3	14	10	31		7.3	FILL
Spun casing												
	3D	2"	24"	1"	12.7	26	16	36	67			~~Granite Blocks,~~ BOULDERS/COBBLES w/Brown Sandy Silt, w/Trace of Gravel
	1R	3"	3.3'	2.5'	15.8	75.8%						NUMEROUS COBBLES & BOULDERS RANGING IN THICKNESS FROM 0.3' TO 1.2'.
	4D	2"	24"	0"	17.8	26	11	12	13		18.3	FILL
												~~Rollercone Refusal @ 18.3'~~ DIAMOND SPIN SHOE BROKE IN BOREHOLE - BORING ABANDONED.

SAMPLES
D = SPLIT SPOON, R = ROCK CORE
C = 2" SHELBY TUBE
S = 3" SHELBY TUBE, V = VANE

SOIL CLASSIFIED BY:
XX DRILLER-VISUALLY
SOIL TECHNICIAN-VISUALLY
LABORATORY TESTS

REMARKS:
Spun casing from 7.3' to 16.5';
Rollerconed from 16.5' to 18.3'

HOLE NO. B-2

MAINE TEST BORINGS, INC. BREWER, MAINE 04412	CLIENT GEI Consultants, Inc.	SHEET 1 OF 1 HOLE NO. B-3	
DRILLER Ron Idano	PROJECT NAME Simpson Rd. Bridge	LINE & STATION	
M. [illegible] JOB NUMBER 05-008	LOCATION Saco, ME	OFFSET	

GROUND WATER OBSERVATIONS
[illegible]/11/05 1:00pm Water @ 21.6' w/18.5' Casing, Hole depth= 23.2';
[illegible]/11/05 1:38pm Hole Caved & Dry @ 17.1'

	CASING	SAMPLER	CORE BARREL	DATE	DATE
TYPE	NW	SS	NQ2	01/10/05 Start	01/11/05 Finish
SIZE I.D.	3"	1 3/8"	2"	SURFACE ELEVATION	
HAMMER WT.	300#	140#			
HAMMER FALL	16"	30"			

CASING BLOWS PER FOOT	SAMPLE NO.	O.D.	PEN.	REC.	DEPTH @ BOT.	BLOWS 0-6	6-12	12-18	18-24
Auger	1D	2"	1"	1"	0.6	50/1"			
17	2D	2"	24"	5"	7.0	4	45	22	26
[illegible]									
[illegible]									
61									
22	3D	2"	11"	0	10.9	22	100/5"		
[illegible]0"									
	1R	3"	0.9'	0.9'	12.2	100%			
	2R	3"	0.5'	0.5'	12.7	100%			
	4D	2"	4"	0"	13.0	50/4"			
	3R	3"	0.9'	0.5'	13.9	56%			
	5D	2"	24"	0"	15.9	18	17	47	26
	6D	2"	24"	0"	17.9	17	44	33	37
	4R	3"	3.9'	3.7'	22.4	95%			
	5R	3"	3.6'	3.5'	26.0	97%			

DEPTH	STRATUM DESCRIPTION
0.5	Pavement
10.9'	BROWN SILTY SAND & Brown Sandy Silt w/Gravel CONTAINING LUMPS OF SILTY CLAY AND COBBLES/BOULDERS. FILL
18.5'	~~Granite Blocks,~~ BOULDERS/COBBLES w/Brown Sandy Silt, w/Trace of Gravel FILL
21.0'	MASONRY BLOCKS
26.0	BEDROCK - METAMORPHIC, FINE-GRAINED, FOLIATED, JOINTS DIPPING AT 0° & ~50°, GRAY, (SCHIST). RQD=60%
	Bottom of Boring @ 26.0'

SAMPLES
D = SPLIT SPOON
C = 2" SHELBY TUBE
S = 3" SHELBY TUBE
R = ROCK CORE
V = VANE

SOIL CLASSIFIED BY:
XX DRILLER-VISUALLY
SOIL TECHNICIAN-VISUALLY
LABORATORY TESTS

REMARKS:
~~100" for 0.8"~~
Spun casing from 10.9' to 18.5' - ABOUT 20 GPM PUMPED WITHOUT RETURN AT TOP OF CASING, BUT V. LITTLE WATER SEEPING FROM FACE OF ABUTMENT [illegible]

HOLE NO. B-3

MAINE TEST BORINGS, INC.
BREWER, MAINE 04412

CLIENT: GEI Consultants, Inc.

SHEET 1 OF 1

HOLE NO. ~~B2~~ B4

DRILLER: Brad Enos

PROJECT NAME: Simpson Road Bridge

LINE & STATION:

M.T.B. JOB NUMBER: 05-008A

LOCATION: Saco, ME

OFFSET:

GROUND WATER OBSERVATIONS

	CASING	SAMPLER	CORE BARREL
TYPE	NW	SS	NQ2
SIZE I.D.	3"	1 3/8"	2"
HAMMER WT.	300#	140#	
HAMMER FALL	16"	30"	

DATE Start: 02/16/05 — DATE Finish: 02/16/05

SURFACE ELEVATION:

CASING BLOWS PER FOOT	SAMPLE NO.	O.D.	PEN.	REC.	DEPTH @ BOT.	BLOWS PER 6" ON SAMPLER 0-6	6-12	12-18	18-24	VANE READING	DEPTH	STRATUM DESCRIPTION
Auger											0.8'	Pavement & Concrete
												Brown Silty Sand w/Gravel
Spun	1D	2"	13"	12"	6.1	7	13	50/1"			6.1'	FILL
Casing												
												Boulders & Cobbles w/Brown Silty Gravelly Sand & SANDY SILT.
	R	3"	3.0'	0.0'	13.3	0%						MOSTLY COBBLES & BOULDERS RANGING IN THICKNESS FROM 0.7' TO 2.5'.
	R	3"	5.0'	0.0'	18.3	0%						
	R	3"	5.0'	5.0'	20.0	0%					19.7'	FILL
	1R	3"	1.8'	1.8'	21.5	100%					21.5'	BEDROCK - METAMORPHIC, FINE-GRAINED, FOLIATED, JOINTS AT 1" TO 3" SPACINGS DIPPING Rock AT 0° AND ~70°. (SCHIST) RQD = 0%. GRAY
												Bottom of Boring @ 21.5'

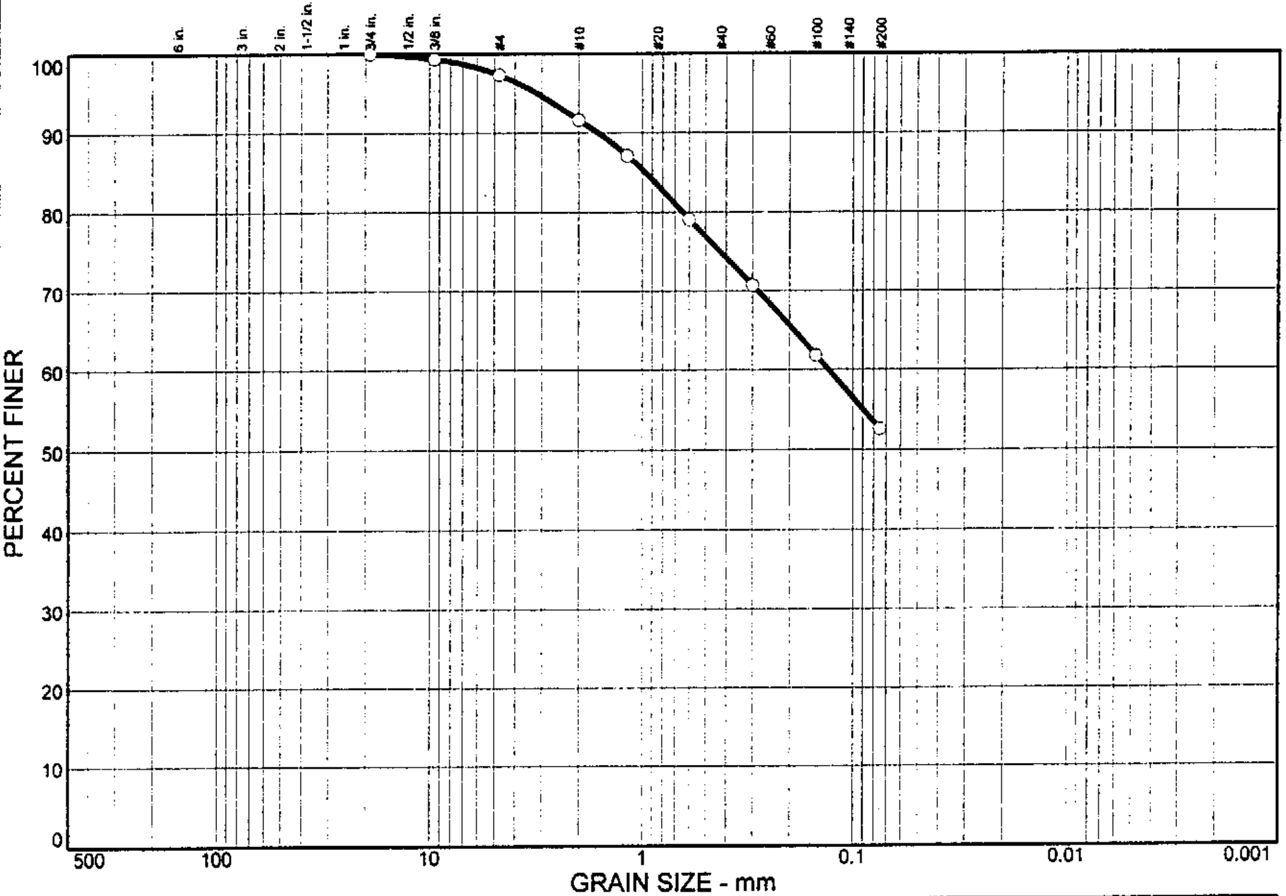
SAMPLES
D = SPLIT SPOON — R = ROCK
C = 2" SHELBY TUBE — CORE
S = 3" SHELBY TUBE — V = VANE

SOIL CLASSIFIED BY:
- XX DRILLER-VISUALLY
- SOIL TECHNICIAN-VISUALLY
- LABORATORY TESTS

REMARKS:
Attempted spinning NW casing-Broke shoe @ 10.0'

HOLE NO. ~~B2~~ B4

GRAIN SIZE DISTRIBUTION TEST REPORT



% + 3"	% GRAVEL	% SAND	% SILT	% CLAY
0.0	2.7	44.7	52.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.75 in.	100.0		
.375 in.	99.3		
#4	97.3		
#10	91.6		
#16	87.1		
#30	79.1		
#50	70.7		
#100	61.8		
#200	52.6		

* (no specification provided)

Soil Description

Sandy CLAY, olive
OR SANDY SILT

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.972 D_{60}= 0.131 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

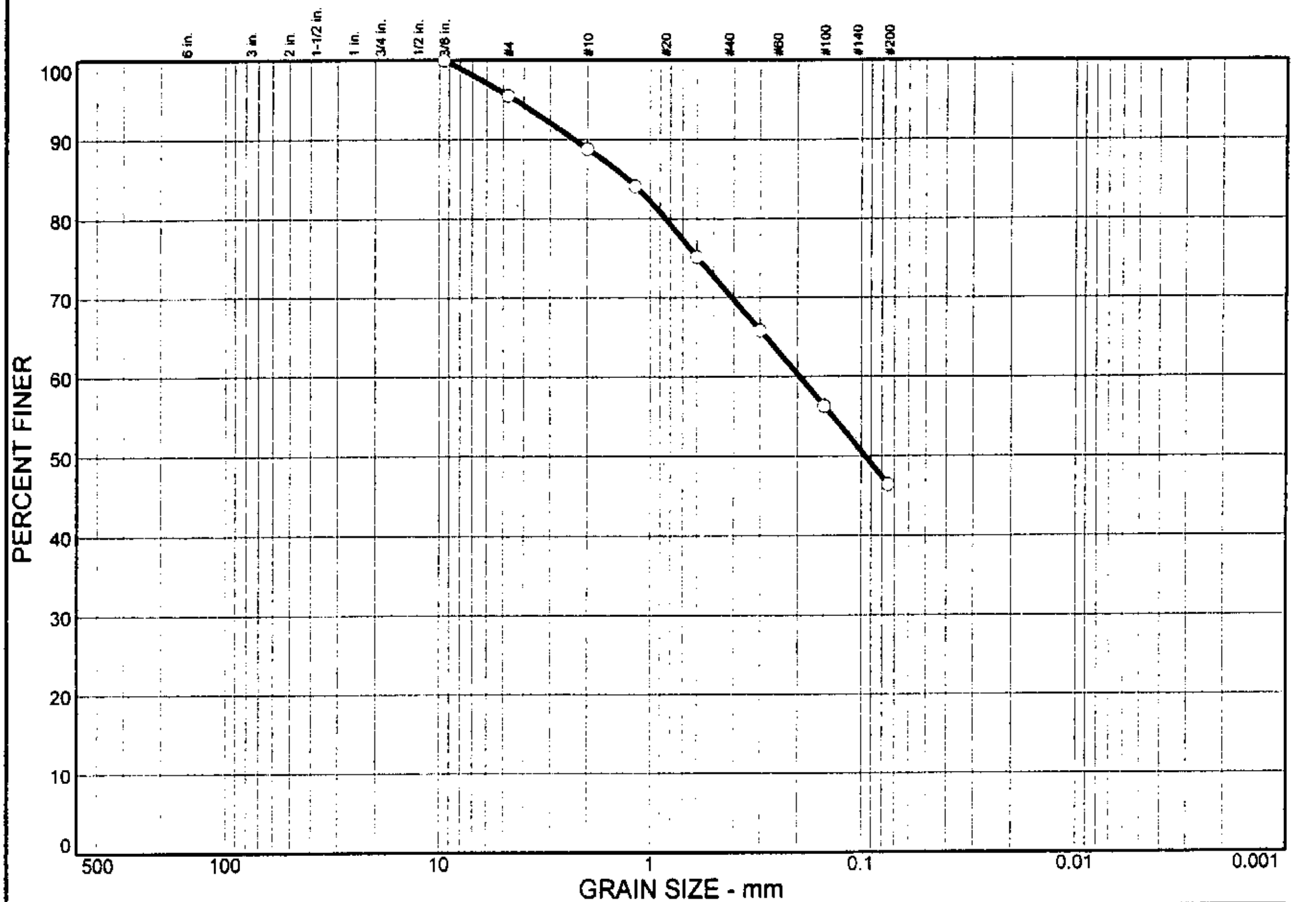
Date tested: 2-23-05

Sample No.: S1 **Source of Sample:** B1 **Date:**
Location: **Elev./Depth:** 5-7 ft

GEI Consultants

Client: CLD Consulting Engineers, Inc.
Project: Simpson Road Bridge
Project No: 050030 **Fig.**

GRAIN SIZE DISTRIBUTION TEST REPORT



% + 3"	% GRAVEL	% SAND	% SILT	% CLAY
0.0	4.5	49.1	46.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375 in.	100.0		
#4	95.5		
#10	88.8		
#16	84.1		
#30	75.2		
#50	65.8		
#100	56.3		
#200	46.4		

* (no specification provided)

Soil Description

Silty SAND, olive

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.29 D_{60}= 0.196 D_{50}= 0.0963
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

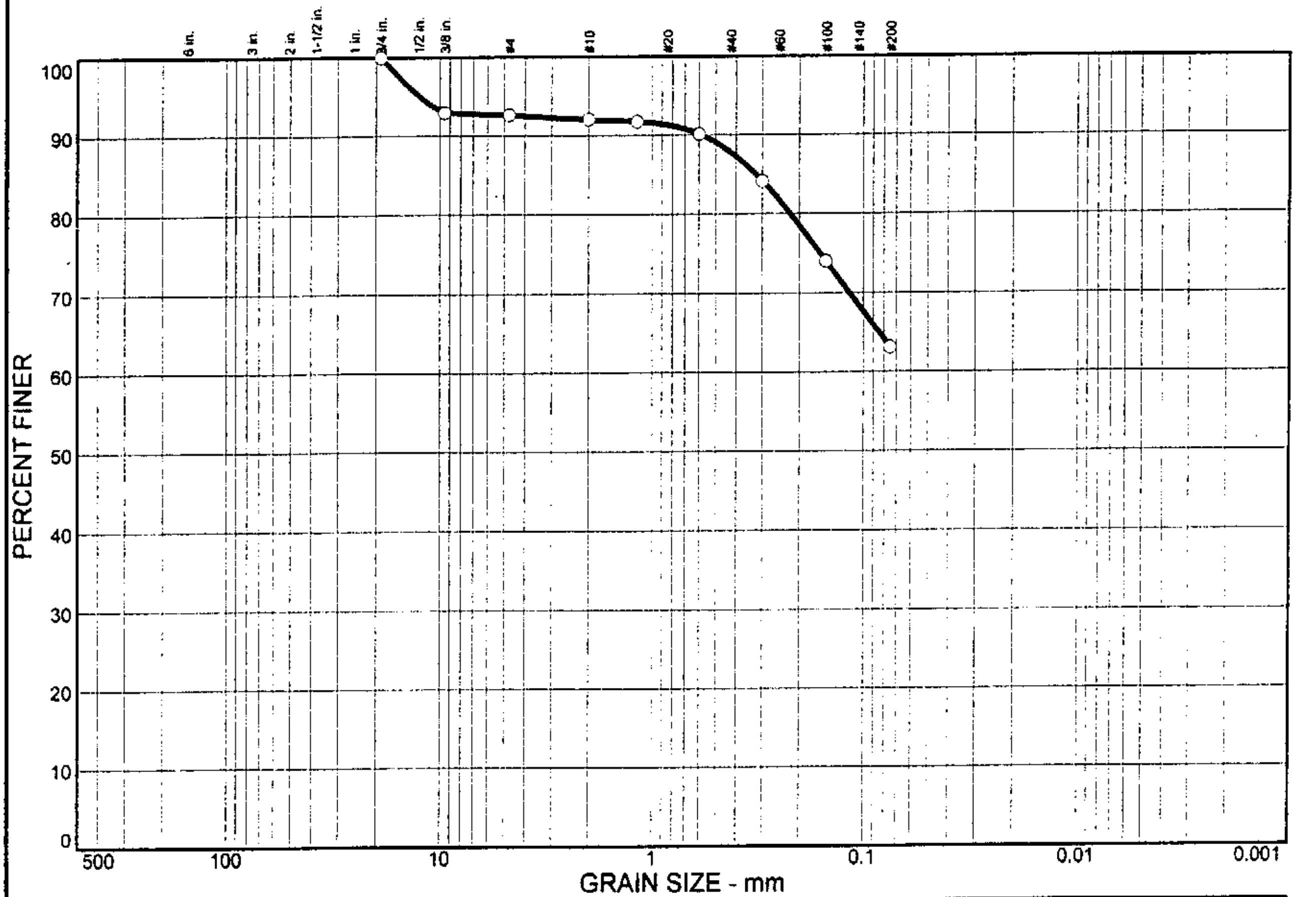
Date tested: 2-23-05

Sample No.: A3 **Source of Sample:** B1 **Date:**
Location: **Elev./Depth:** 3-4 ft

GEI Consultants

Client: CLD Consulting Engineers, Inc.
Project: Simpson Road Bridge
Project No: 050030 **Fig.**

GRAIN SIZE DISTRIBUTION TEST REPORT



% + 3"	% GRAVEL	% SAND	% SILT	% CLAY
0.0	7.4	29.5	63.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.75 in.	100.0		
.375 in.	92.9		
#4	92.6		
#10	92.0		
#16	91.7		
#30	90.1		
#50	84.1		
#100	74.0		
#200	63.1		

* (no specification provided)

Soil Description

Sandy CLAY, olive

OR SANDY SILT

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.324 D_{60}= D_{50}=

D_{30}= D_{15}= D_{10}=

C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Date tested: 2-23-05

Sample No.: -- **Source of Sample:** B4 **Date:**

Location: **Elev./Depth:** 19-19.7 ft

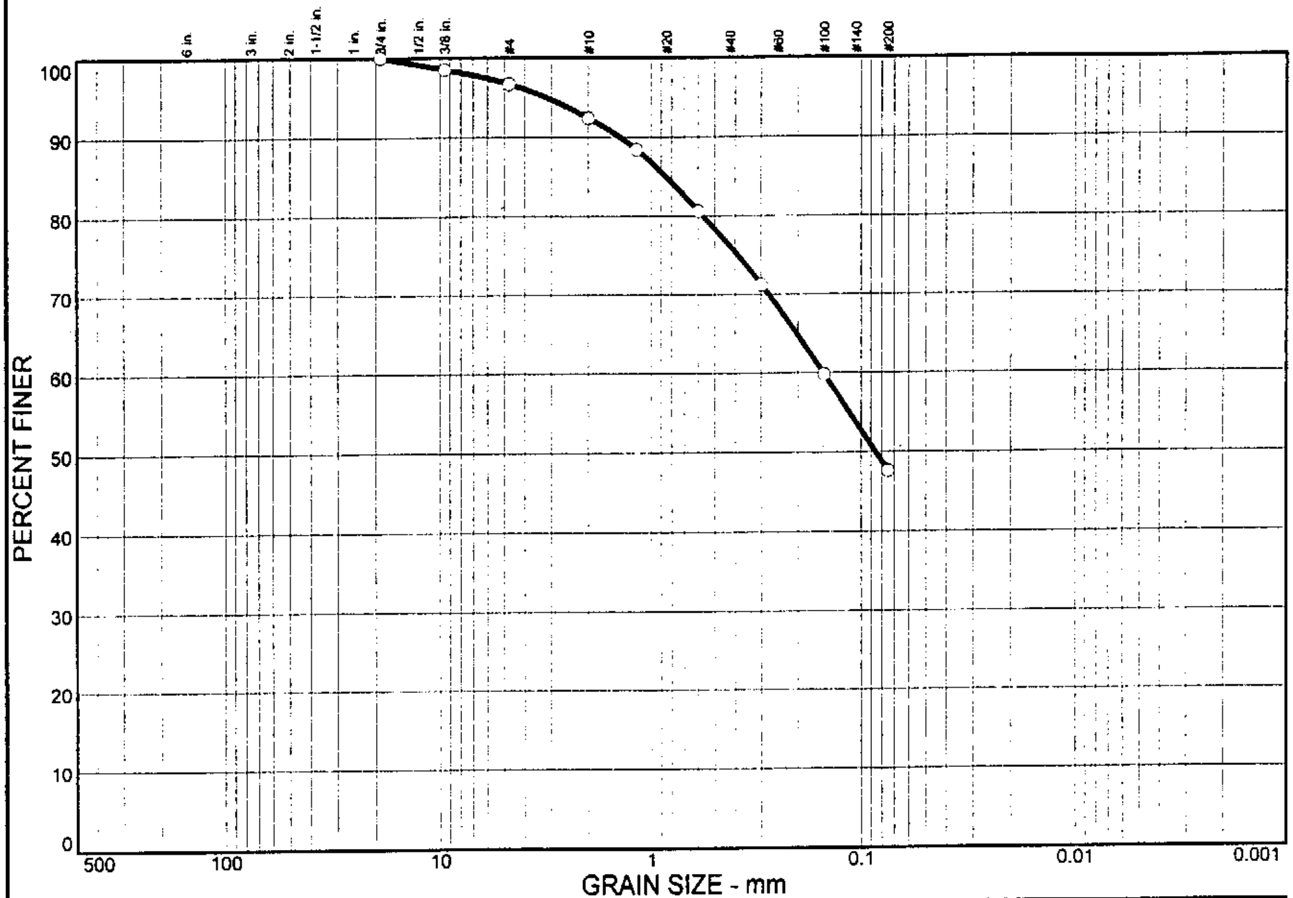
GEI Consultants

Client: CLD Consulting Engineers, Inc.

Project: Simpson Road Bridge

Project No: 050030 **Fig.**

GRAIN SIZE DISTRIBUTION TEST REPORT



% + 3"	% GRAVEL	% SAND	% SILT	% CLAY
0.0	3.3	49.1	47.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.75 in.	100.0		
.375 in.	98.5		
#4	96.7		
#10	92.4		
#16	88.4		
#30	80.6		
#50	71.2		
#100	59.8		
#200	47.6		

* (no specification provided)

Soil Description

Silty SAND, olive

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.857 D_{60}= 0.152 D_{50}= 0.0859
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Date tested: 2-23-05

Sample No.: S1 **Source of Sample:** B4 **Date:**
Location: **Elev./Depth:** 5-6.1 ft

GEI Consultants

Client: CLD Consulting Engineers, Inc.
Project: Simpson Road Bridge
Project No: 050030 **Fig.**

KICK GEOEXPLORATION
Geophysical & Geological Investigations
348 Pleasant Street, Box 6, Dunstable, MA 01827

April 13, 2005

Mr. Craig F. Ward, P.E.
GEI Consultants, Inc.
53 Regional Drive
Concord, New Hampshire 03301

Re: Ground Penetrating Radar Survey
Simpson Road Bridge Project
Saco, Maine

Dear Craig:

In accordance with your authorization, Kick Geoexploration and Radar Solutions International (RSI) conducted a ground penetrating radar survey at the above-referenced site. The purpose of the survey was to determine the thicknesses of the granite masonry walls that support the approach embankment. President and Sr. Geophysicist, Ms. Doria Kutrubes, Engineer Ms. Sumeet Chani, and Geophysicist Margela Andrews of RSI and Dr. John Kick, President and Sr. Geophysicist of Kick Geoexploration conducted the survey. Field work for the survey was completed on February 1st and 2nd, 2005.

LOCATION AND SURVEY CONTROL

The Simpson Road bridge is located in Saco, Maine, on Simpson Road where it crosses Stackpole Creek. A baseline was established parallel to the long axis of the bridge to determine the location of the radar transects. Points along the baseline were measured using a fiberglass tape and referenced to a utility pole on the southwest side of the bridge. (See site map, Figure 1). Work progress was slowed considerably by difficult access along the sides of the bridge because of steep and slippery embankments and deep snow.

METHODOLOGY

RSI used a state-of-the-art GSSI SIR 3000 digital radar system with a 400 MHz antenna to collect data. The radar antenna propagates high frequency radio and microwave energy in to the ground, where this energy is reflected back to the antenna by materials of differing electrical (i.e. dielectric and conductivity), and to a lesser extent, physical properties. Ground penetrating radar (GPR) provided a pseudo cross-section wherever the antenna was moved. For this survey, the GPR survey lines were oriented vertically, and the antenna was moved, with the assistance of an extension rod, from the top of the wall to the base of the wall. The horizontal scale of the output GPR record corresponds to the vertical position of the antenna as it is moved along the survey line. The vertical scale of the GPR record represents two-way travel time, out to and reflected from the back of the wall. The time-scale is converted to a depth by estimating the velocity of the EM waves through the medium. For this survey, a velocity of 0.35 ft/ns was estimated.

The wall of the bridge is made of discrete granite blocks that likely form a series of 'steps' in the interior of the wall as the wall widens downward. One would therefore expect to see a corresponding 'stepped' pattern in the radar images but this does not occur. The radar antenna propagates waves in a clover-shaped radiation pattern that detects not only the backs of the in-line blocks but also adjacent blocks. The result is a smoothed image and in a total scan the back of the wall appears as an inclined (dipping) line.

GPR data were acquired along lines spaced approximately 5 feet apart and situated along both north and south walls. As Stackpole Creek bisects both walls, for purposes of this report, each portion of the wall surveyed was assigned a quadrant designation. Figure 1 shows the horizontal position of GPR survey lines, as referenced to a utility pole located southwest of the south wall, and to the top of the wall. Station 0 along each vertical transect corresponds to the top of the wall.

Data were then transferred to desktop computer where it was interpreted. A more detailed discussion of GPR theory and its limitations is found in Appendix A.

RESULTS

Figures 2 through 11 summarize our GPR interpretation. All figures are presented at a scale of 1 inch = 2 feet. A number of different reflectors were observed and plotted on the figures and labeled in the legend. Reflection signal patterns can be complex and difficult to understand. The interpreter's task is to characterize the various reflection patterns and determine which is significant. The first row of blocks was typically viewed within 1.5 to 2 feet, and was generally the first reflection observed in the time/depth record. The first row of blocks is denoted by a magenta-dashed line on the accompanying figures.

Two inclined reflectors were typically observed slanting away from the outer (vertical) surface of the wall as the antenna was moved toward the base of the wall. The shallowest of these inclined reflectors is denoted as a solid blue line and represents the minimum possible thickness of the wall. The green-dashed line is the second, deeper inclined reflector, and likely represents the maximum wall thickness. Reflections patterns interpreted to represent the back of the wall,and which therefore allow the determination of thickness, is shown as a black dashed line on the accompanying figures. Reflections possibly indicating a void behind the blocks is shown as yellow diamonds on the attached figures. Key results are summarized below.

Please note, when viewing the figures, that they are graphs and do not a directly represent wall shape. Departures from verticality of the outer wall of the bridge, as well as smoothing effects from the antenna radiation pattern, could well be manifested in the interpreted shape of the back wall. An example GPR record is found in Appendix B.

Discussion of Quadrants

Southwest Quadrant (Figures 2 -3)
The interpreted thickness of the bridge wall increases eastward at the top and bottom, towards the bridge center. The minimum of thickness of the wall at the base of lines S50E and S58E on figure 3 is about 8 feet. The wall, at its thinnest point (i.e along Line S30E) is about 3 feet in thickness.

Southeast Quadrant (Figures 4 - 6)
Variations in bridge wall thickness in the southeast quadrant are similar to what is found in the southwest quadrant but the thickness of the wall at the top appears to be greater in the southeast. The maximum thickness of the wall at the base along Lines S95E through S105E is about 8 to 9 feet, again, similar to that observed in the southwest quadrant.

Northwest Quadrant (Figures 7-9)
Considerable surface irregularity in the form of protruding boulders, cavities and a general bulging were visible on the northwest wall in the field. The irregularities, seen in the field, are manifested on the radar images and displayed on the figures as irregularities and relatively thick sections (considerable changes in thickness with distance). Some irregularities may also be attributed to voiding behind the wall blocks or within the wall itself.

Northeast Quadrant (Figures 10-11)
The interpreted wall thicknesses in the northeast quadrant appear consistent with the other areas. Lines N84E and N90E on figure 10 show walls about five feet thick at the top

widening to up to 10 feet thick at the bottom. As the east end of the bridge is approached the walls thin to less than four feet.

SUMMARY

Both walls appear to have similar dimensions and thicknesses. At the edges of the wall, the abutment thickness ranges from 3.5 to 4 feet, while at the deepest, GPR survey results indicate that the wall is 8 to 10 feet in thickness.

***

We appreciate this opportunity to work with GEI, Inc. Please call should you have any inquiries regarding this or future assignments.

Sincerely,
Kick GeoExploration

John F. Kick

John F. Kick, Ph.D.
President and Senior Geophysicist

APPENDIX A
GROUND PENETRATING RADAR METHOD OF INVESTIGATION

A GSSI SIR System 2 radar instrument using a 400 megahertz (MHz) antenna was used for the survey. GPR data were collected continuously along survey lines and displayed on a color monitor. GPR data were also simultaneously recorded on hard drive for post-survey processing. The horizontal scale on each GPR record is determined by the antenna speed. Survey stations are recorded on GPR records by pressing a marker button as the antenna's centerline passes each grid node (at 5 foot intervals for this survey). The vertical scale of these radar "cross-sections" is determined by the recording interval, which was 60 nanoseconds (ns) using the 400 MHZ antenna. The recording interval represents the maximum two-way travel time in which data is recorded. This recording interval was selected to be greater than the anticipated maximum two-way travel time during which real GPR reflections might be observed. GPR travel times were converted to depths using an approximate dielectric constant determined from "typical" soil propagation velocities from similar sites.

The GPR method operates by transmitting low-powered microwave energy into the ground. The GPR signal is reflected back to the antenna by materials with contrasting electrical (dielectric and conductive) and physical properties. Metal objects, such as USTs, and pipes typically produce high-amplitude hyperbolic reflections on the GPR records. Sometimes concrete blocks, bricks, and cobbles cause similar signatures on the radar record.

SURVEY LIMITATIONS

GPR signals propagate well in sand and gravel. Conditions such as clay, ash, road salt, and fill saturated with brackish or otherwise conductive groundwater, cause GPR signal attenuation and loss of target resolution (i.e. limited detection of small objects). Typically, when background conductivity measurements exceed 30 millimhos per meter (mmhos/m), GPR signal penetration is limited to 1.0 to 1.5 meters. Reinforced concrete also causes limited GPR penetration and resolution. Signal penetration under these conditions is quite variable, ranging from about 3 to 5 feet depending upon the type and spacing of metal reinforcing.

GPR is an interpretive method, based on the subjective identification of reflection patterns that may not uniquely identify a subsurface target or stratigraphic horizon. For instance, the hyperbolic reflector corresponding to a utility is similar in characteristic to that produced by a metal scrap or cobble. Utilities are inferred from where hyperbolic reflectors of similar depth and reflection characteristics align along adjacent lines. Reflections from USTs are asymmetric: reflectors appear flat and of finite dimensions when the antenna moves parallel to the UST's long axis, but appear as large hyperbolic reflectors when the antenna crosses obliquely or perpendicular to the short axis of the UST. In both instances, UST reflectors are of finite length.

Obtaining data along multiple survey traverses help determine the size, shape, and continuity of buried objects. For instance, buried utilities are interpreted from hyperbolic reflectors of similar depth and appearance, which are aligned along adjacent lines. GPR data interpretation is more subjective than that for most other geophysical methods, and confirmation using boreholes or test pits is strongly recommended.

Changes in the speed at which the antenna is moved between stations causes slight errors in horizontal distance interpolations and hence interpreted object positions. Such interpolation errors were minimized by using 5 foot distance marks during this survey and subsequently "rubber sheeting" the data.

The antenna radiation pattern is cone-shaped, emanating GPR signals approximately 15 degrees from horizontal fore and aft, and about 45 degrees from horizontal along the sides, of the antenna, depending upon the dielectric properties of the soil. Therefore, buried objects may be detected before the antenna is located directly over them. GPR anomalies often appear larger than actual target dimensions.

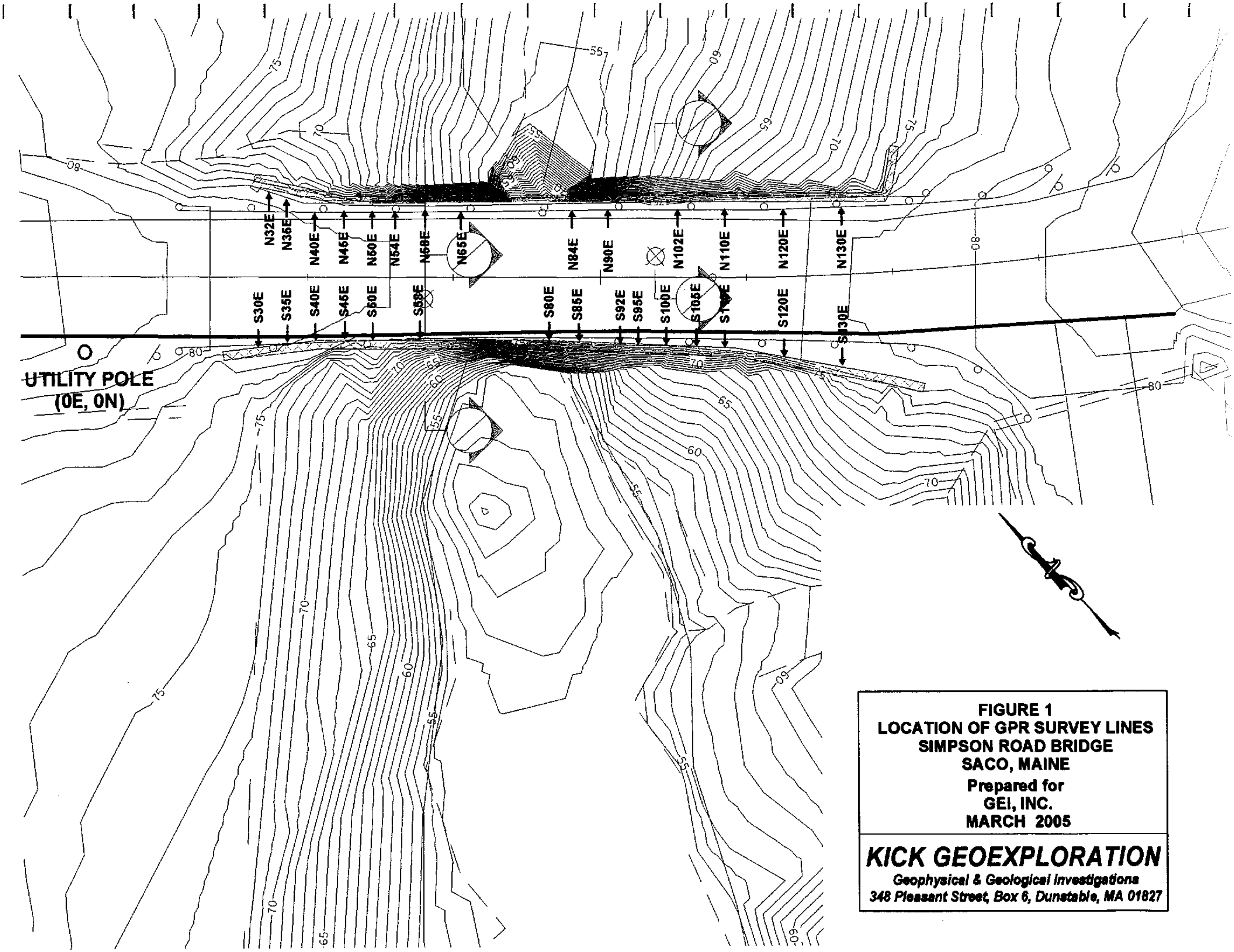
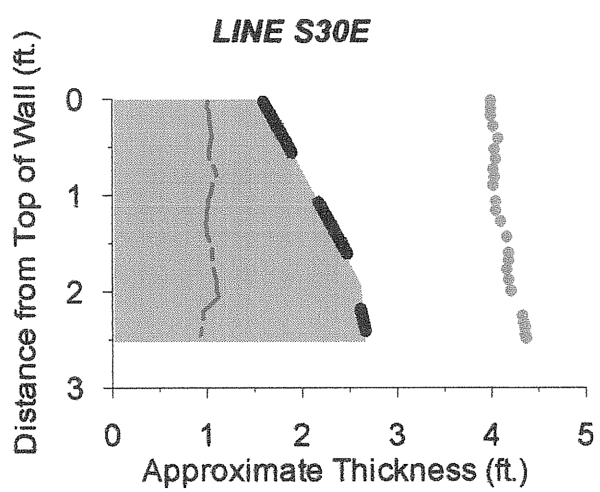
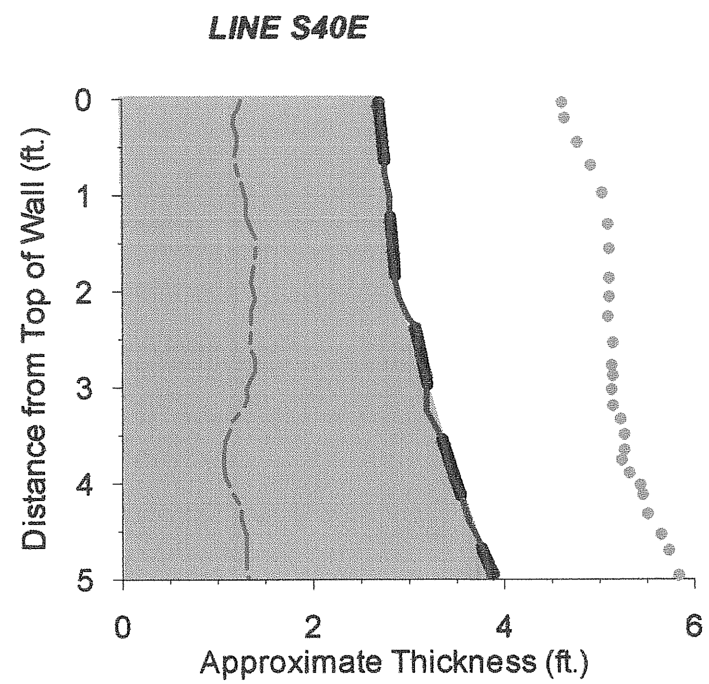
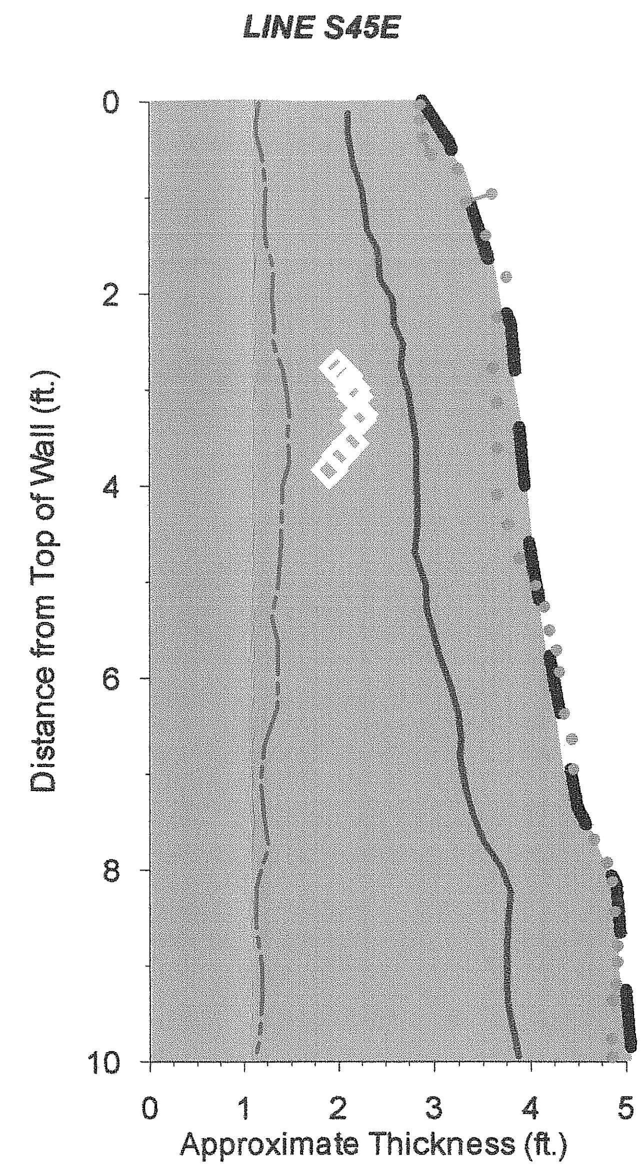


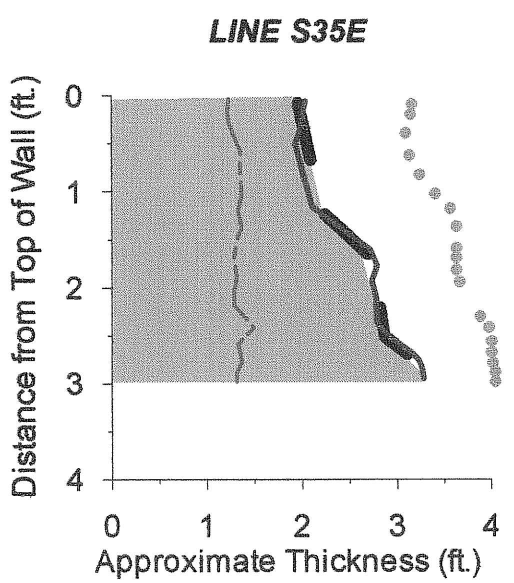
FIGURE 1
LOCATION OF GPR SURVEY LINES
SIMPSON ROAD BRIDGE
SACO, MAINE
Prepared for
GEI, INC.
MARCH 2005

KICK GEOEXPLORATION
Geophysical & Geological Investigations
348 Pleasant Street, Box 6, Dunstable, MA 01827









LEGEND

Minimum thickness estimate (based on first pyramid-shaped reflector)

Reflector attributed to first row of blocks

Maximum thickness of wall (based on deepest pyramid-shaped reflector)

Interpreted Back of Wall (feet)

Interpreted Void or Water within a large Joint

SCALE: 1 Inch = 2 Feet

2 1 0 2 Feet

FIGURE 2
INTERPRETED GPR RESULTS
SOUTHWEST QUADRANT (Q1)
SIMPSON ROAD BRIDGE
SACO, MAINE
Prepared for
GEI CONSULTANTS
APRIL 2005

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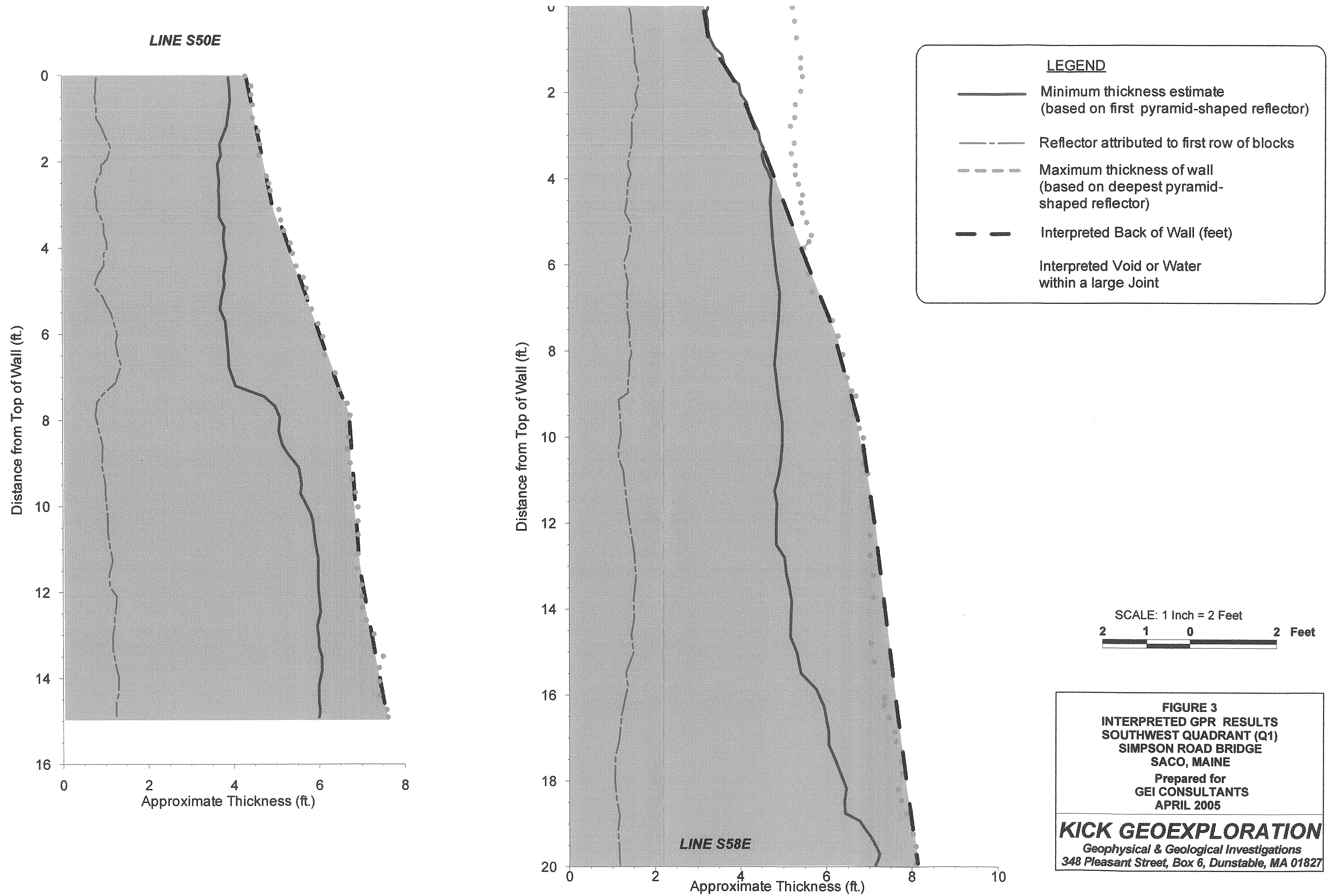

FIGURE 3
INTERPRETED GPR RESULTS
SOUTHWEST QUADRANT (Q1)
SIMPSON ROAD BRIDGE
SACO, MAINE
Prepared for
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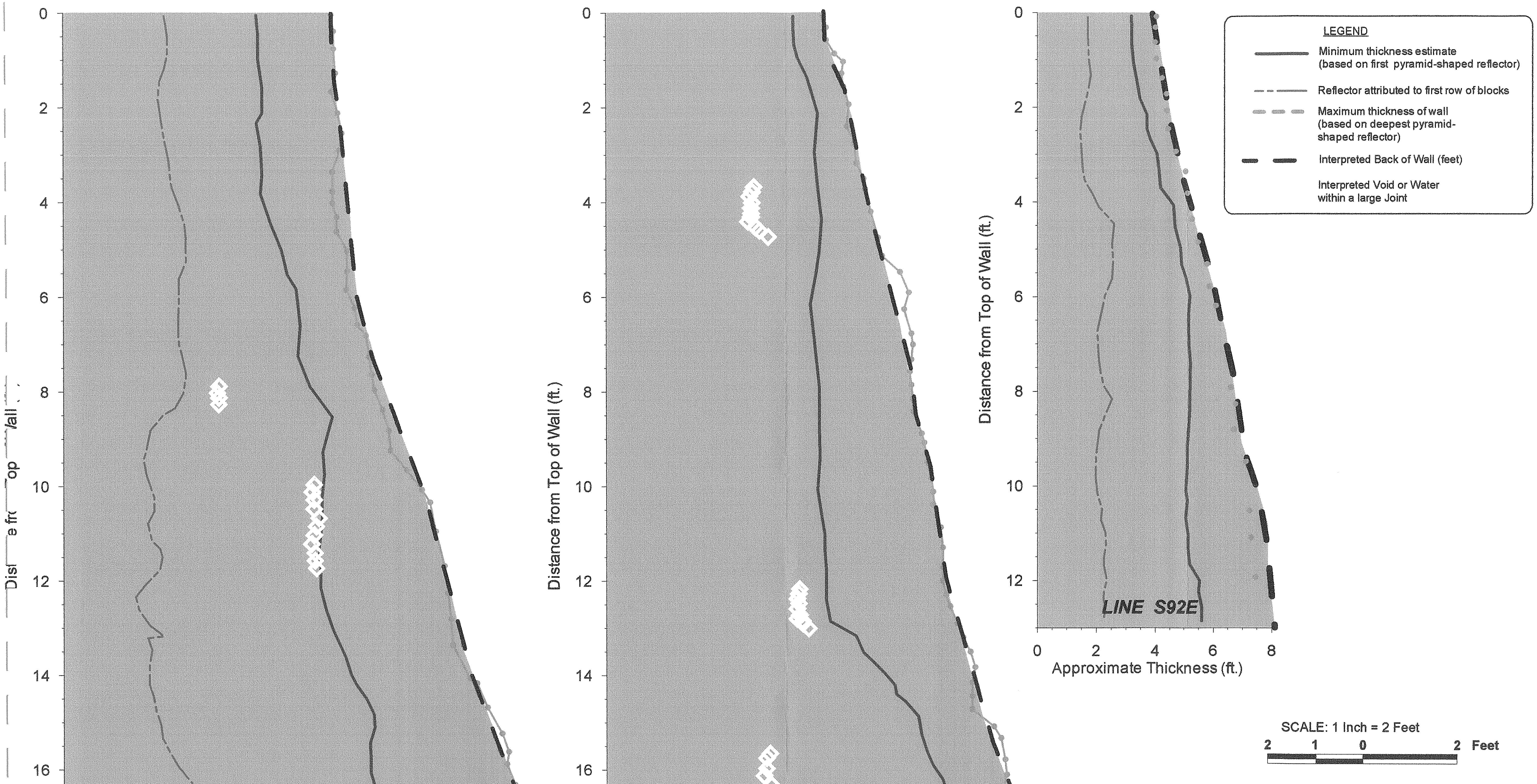

FIGURE 4
INTERPRETED GPR RESULTS
SOUTHEAST QUADRANT (Q2)
SIMPSON ROAD BRIDGE
SACO, MAINE
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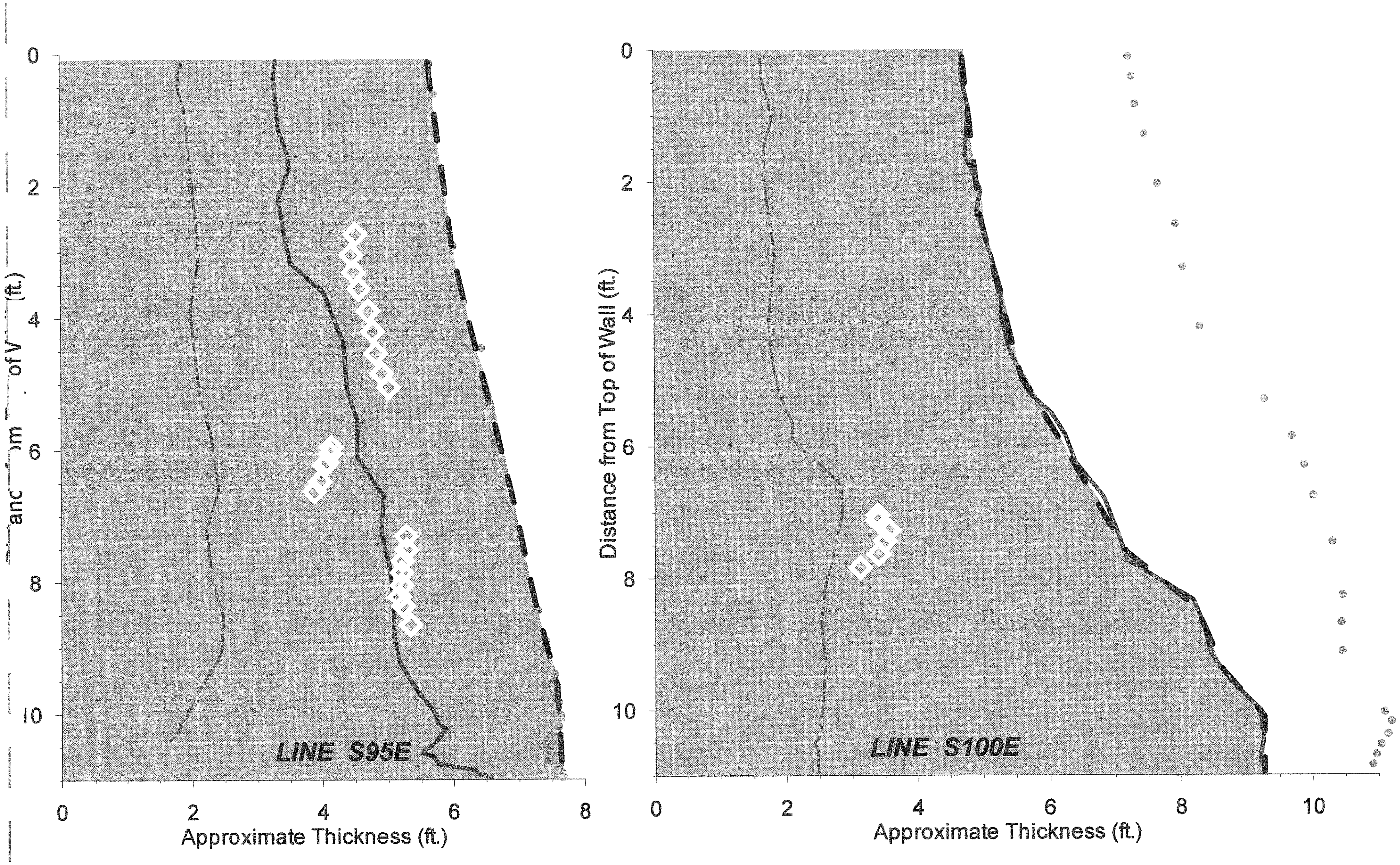

LEGEND

Minimum thickness estimate (based on first pyramid-shaped reflector)

Reflector attributed to first row of blocks

Maximum thickness of wall (based on deepest pyramid-shaped reflector)

Interpreted Back of Wall (feet)

Interpreted Void or Water within a large Joint

SCALE: 1 Inch = 2 Feet

2 1 0 2 Feet

FIGURE 5
INTERPRETED GPR RESULTS
SOUTHEAST QUADRANT (Q2)
SIMPSON ROAD BRIDGE
SACO, MAINE
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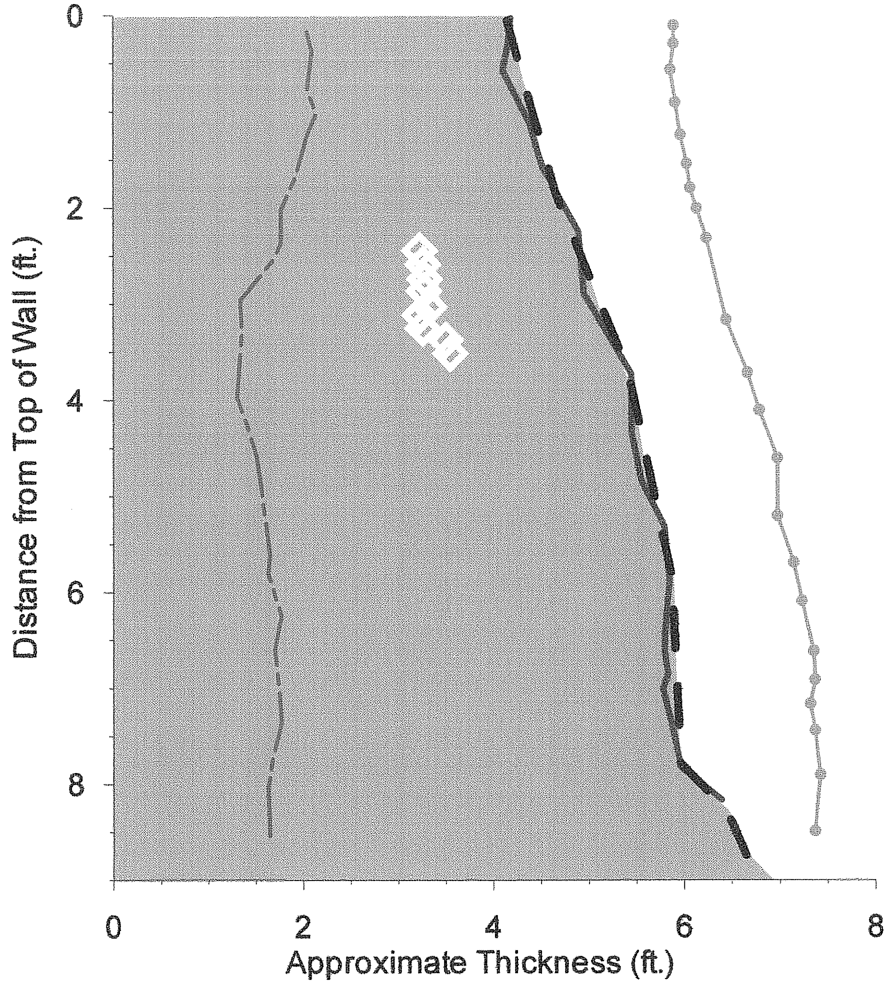
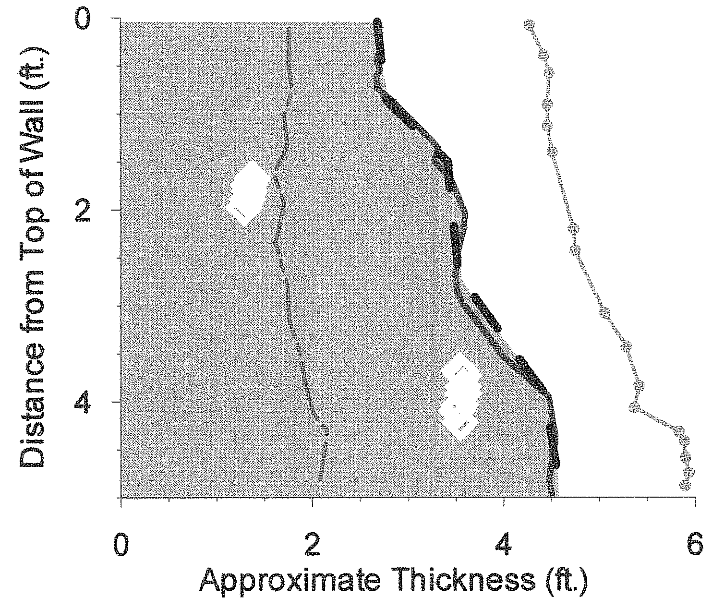


LINE S110E



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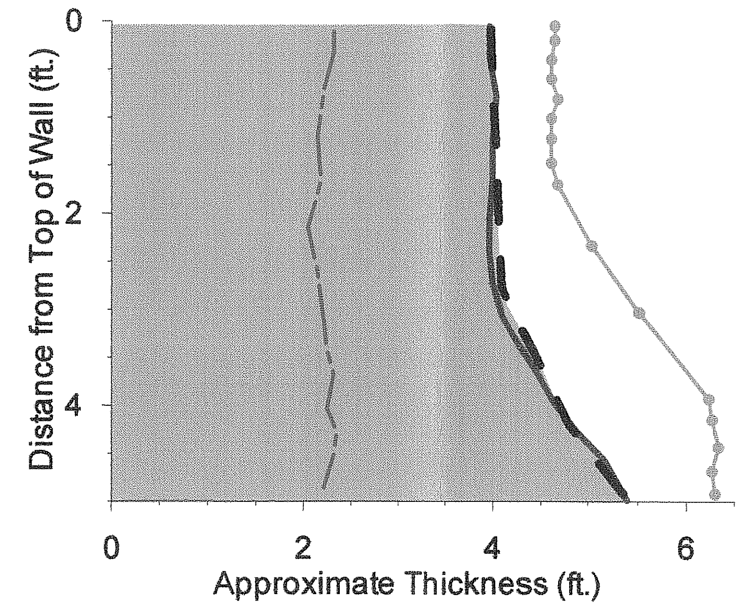

LINE S130E

LEGEND

- Minimum thickness estimate (based on first pyramid-shaped reflector)
- Reflector attributed to first row of blocks
- Maximum thickness of wall (based on deepest pyramid-shaped reflector)
- Interpreted Back of Wall (feet)
- Interpreted Void or Water within a large Joint

SCALE: 1 Inch = 2 Feet

2 1 0 2 Feet

FIGURE 6
INTERPRETED GPR RESULTS
SOUTHEAST QUADRANT (Q2)
SIMPSON ROAD BRIDGE
SACO, MAINE
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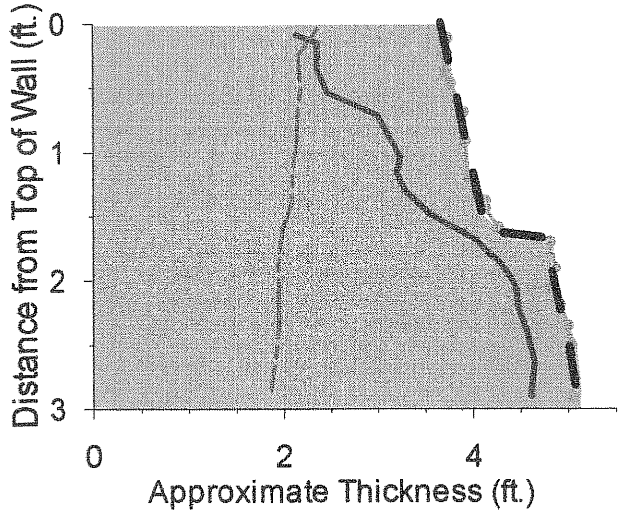
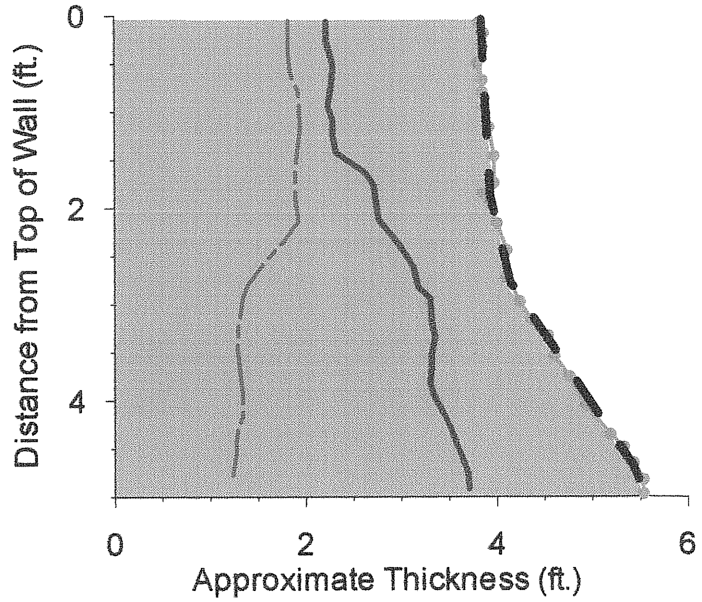
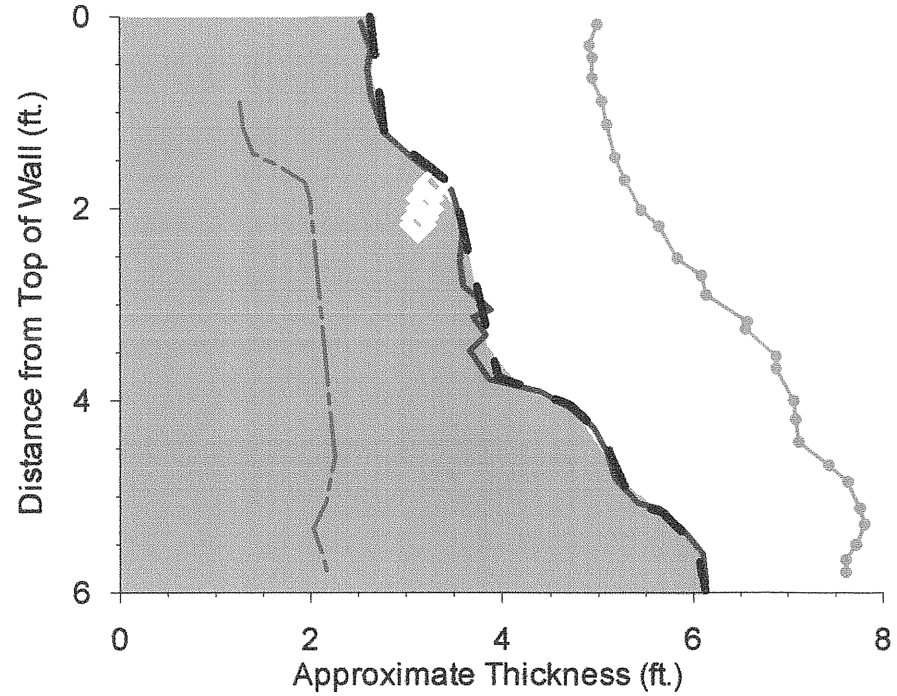


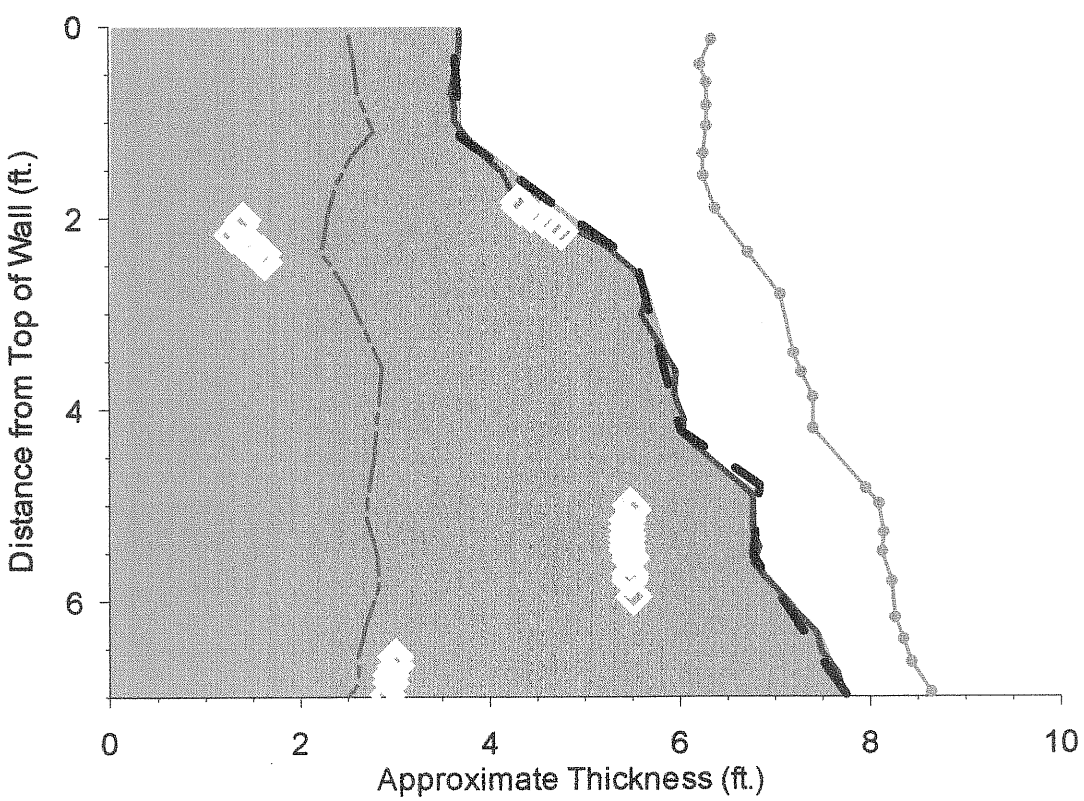
LINE N32E



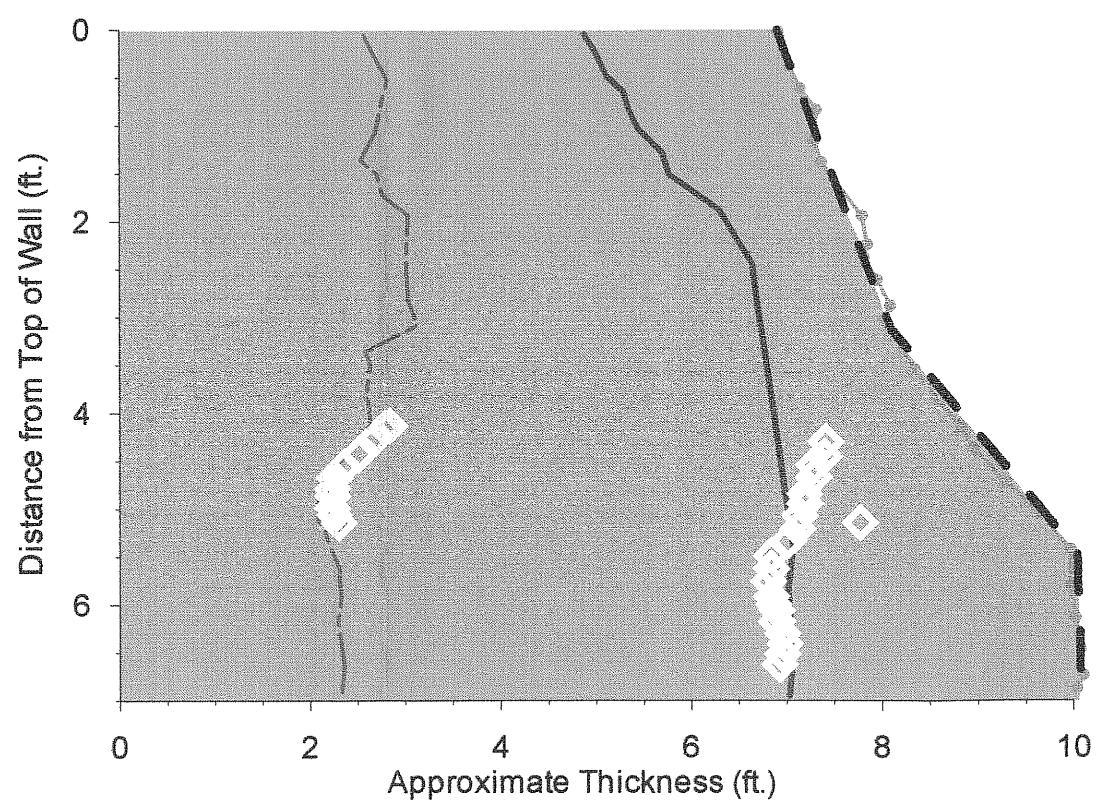
LINE N35E



LINE N40E



LINE N45E



LINE N50E

LEGEND

- Minimum thickness estimate (based on first pyramid-shaped reflector)
- Reflector attributed to first row of blocks
- Maximum thickness of wall (based on deepest pyramid-shaped reflector)
- Interpreted Back of Wall (feet)
- Interpreted Void or Water within a large Joint

SCALE: 1 Inch = 2 Feet

2 1 0 2 Feet

FIGURE 7
INTERPRETED GPR RESULTS
NORTHWEST QUADRANT (Q4)
SIMPSON ROAD BRIDGE
SACO, MAINE
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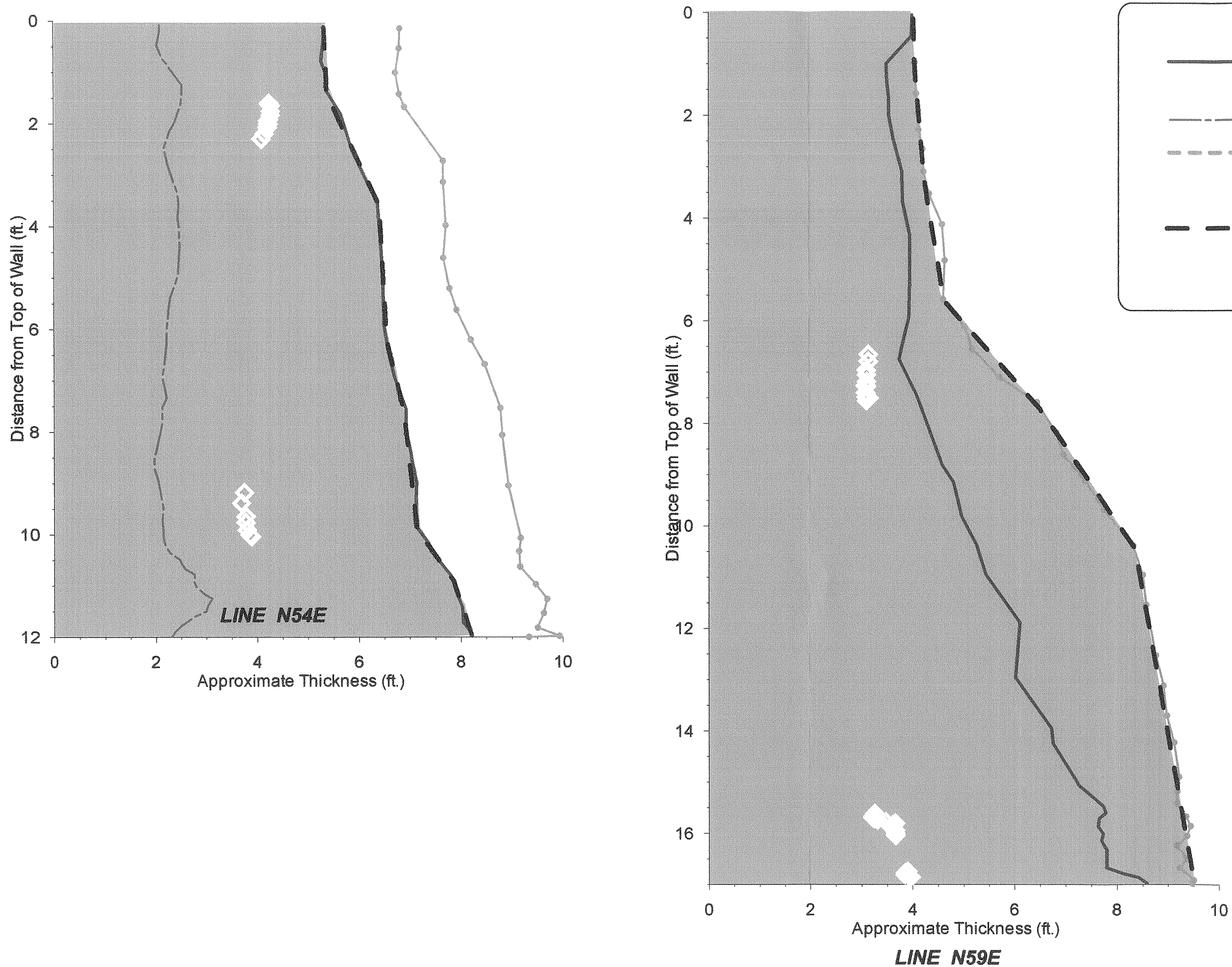

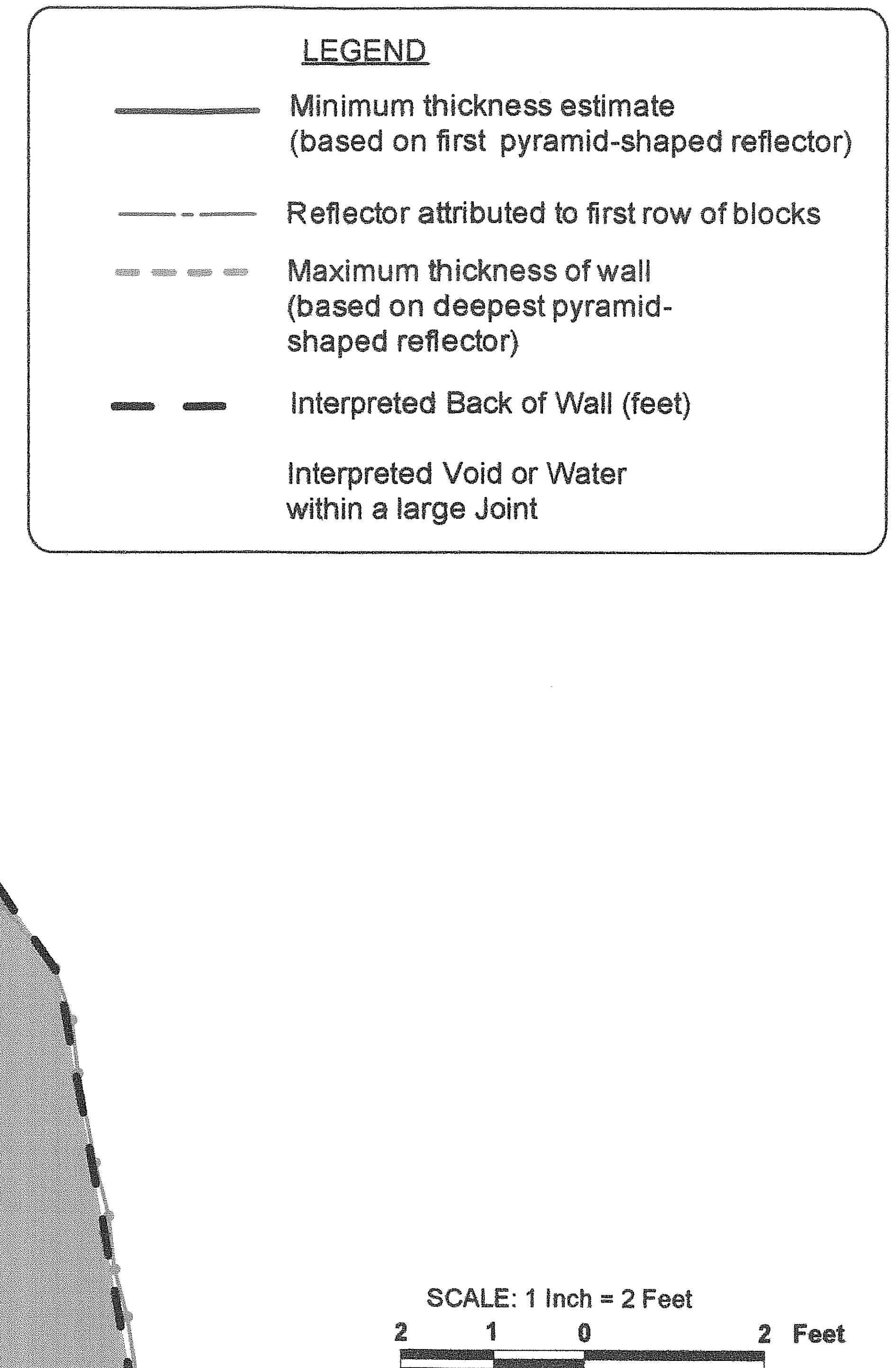

FIGURE 8
INTERPRETED GPR RESULTS
NORTHWEST QUADRANT (Q4)
SIMPSON ROAD BRIDGE
SACO, MAINE
Prepared for
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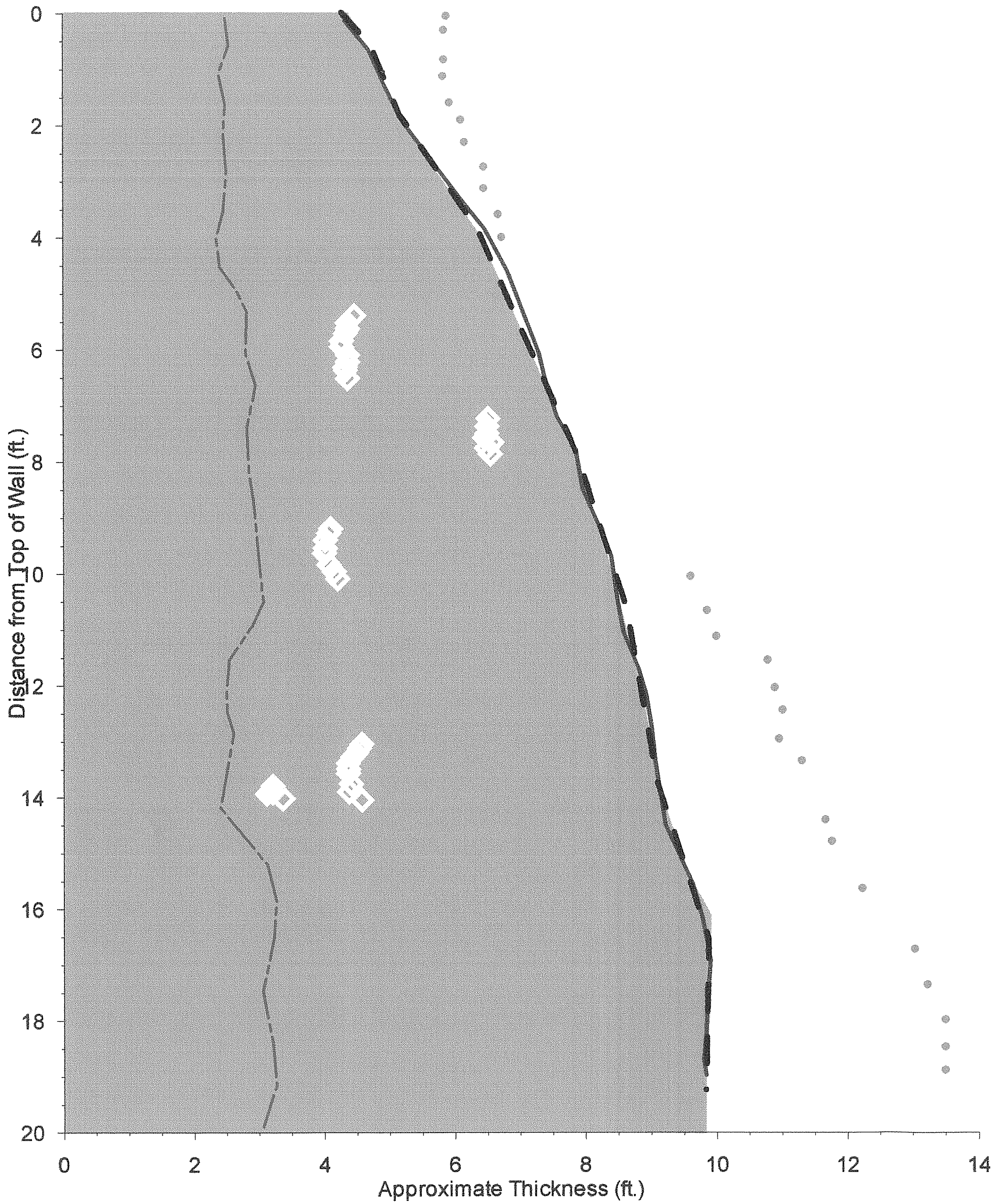

LINE N65E

LEGEND

Minimum thickness estimate (based on first pyramid-shaped reflector)

Reflector attributed to first row of blocks

Maximum thickness of wall (based on deepest pyramid-shaped reflector)

Interpreted Back of Wall (feet)

Interpreted Void or Water within a large Joint

SCALE: 1 Inch = 2 Feet

2 1 0 2 Feet

FIGURE 9
INTERPRETED GPR RESULTS
NORTHWEST QUADRANT (Q4)
SIMPSON ROAD BRIDGE
SACO, MAINE
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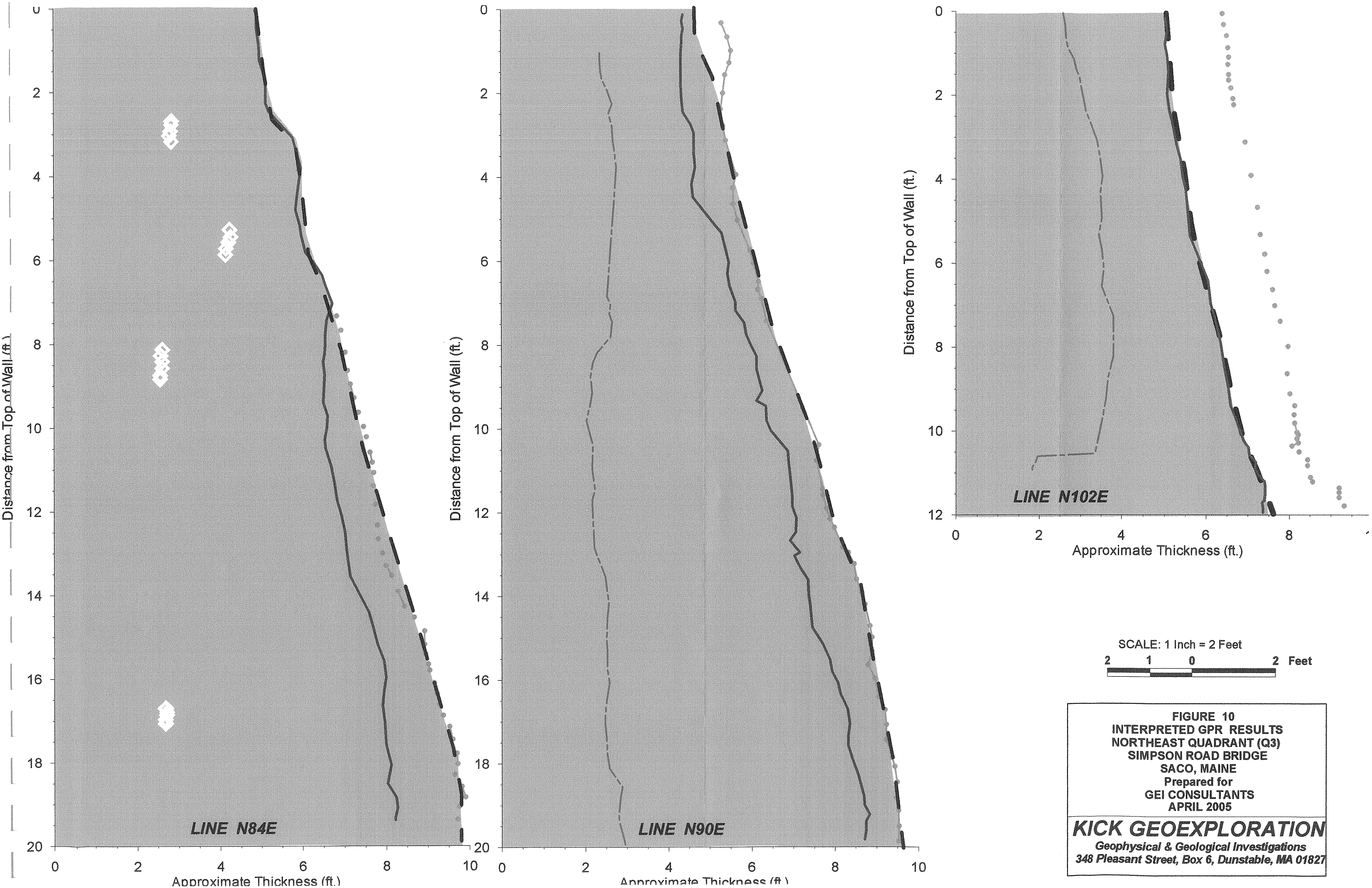

SCALE: 1 Inch = 2 Feet

2 1 0 2 Feet

FIGURE 10
INTERPRETED GPR RESULTS
NORTHEAST QUADRANT (Q3)
SIMPSON ROAD BRIDGE
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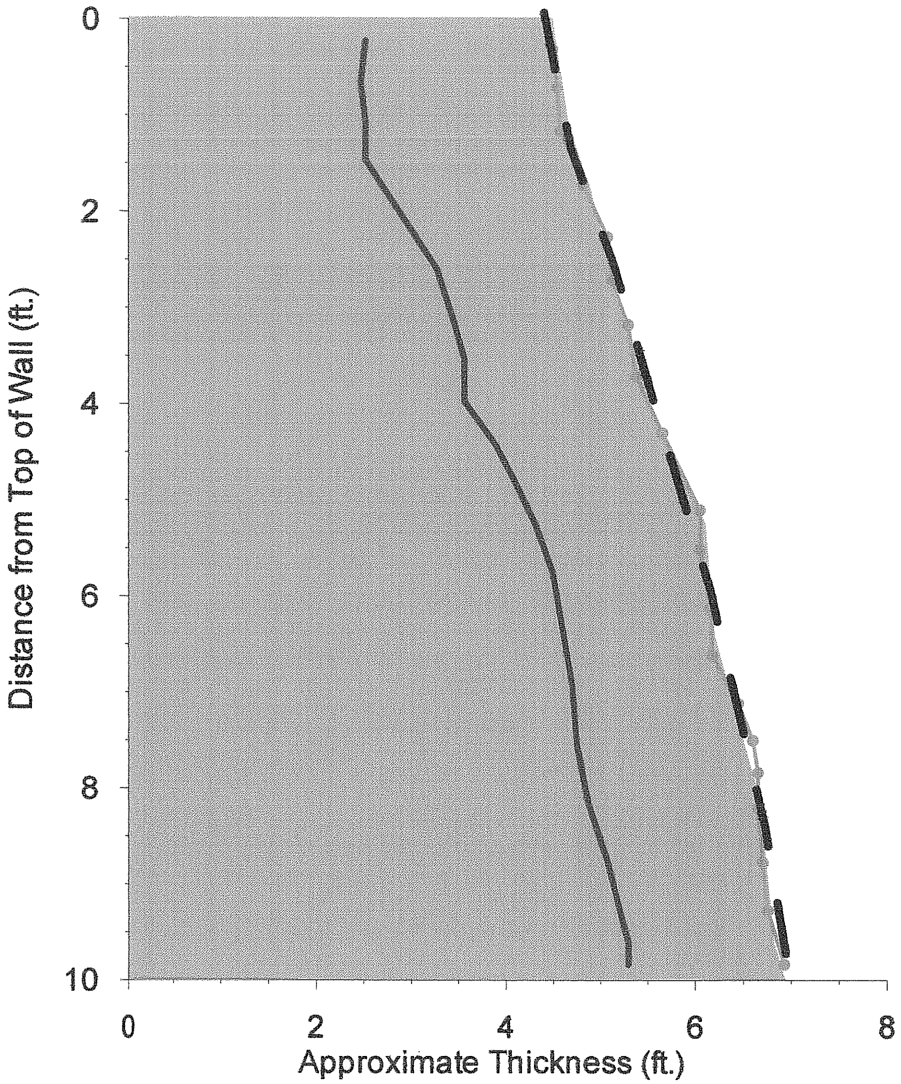
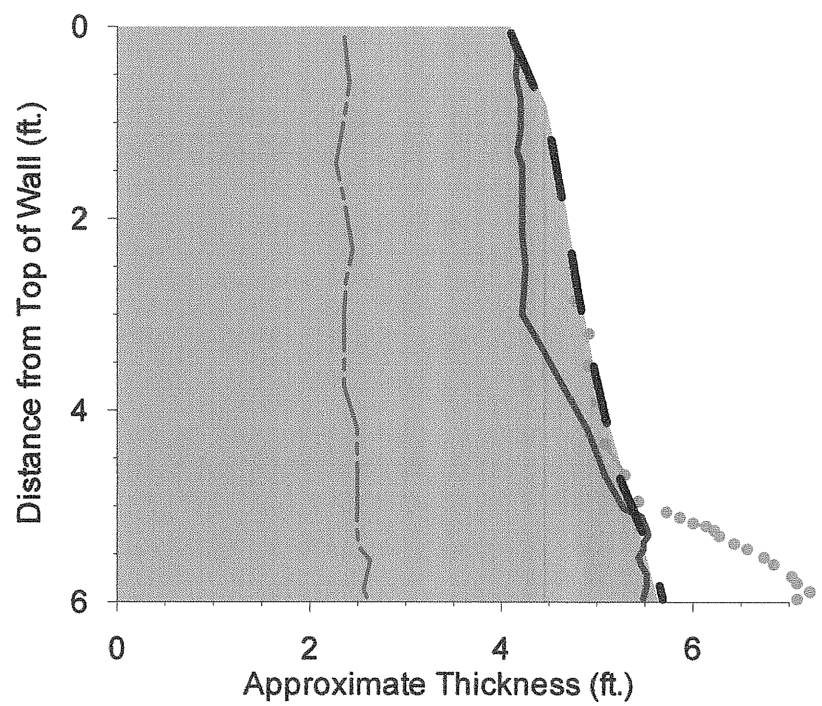
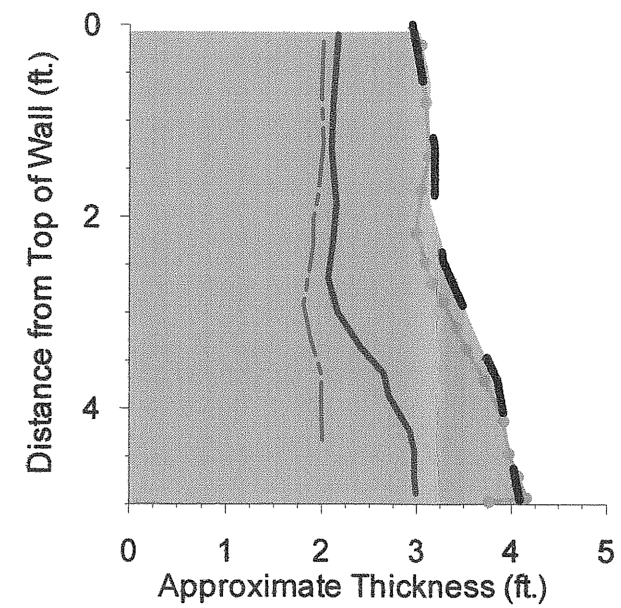
LINE N110E



LINE N120E



LINE N130E



LEGEND

- Minimum thickness estimate (based on first pyramid-shaped reflector)
- Reflector attributed to first row of blocks
- Maximum thickness of wall (based on deepest pyramid-shaped reflector)
- Interpreted Back of Wall (feet)
- Interpreted Void or Water within a large Joint

SCALE: 1 Inch = 2 Feet

2 1 0 2 Feet

FIGURE 11
INTERPRETED GPR RESULTS
NORTHEAST QUADRANT (Q3)
SIMPSON ROAD BRIDGE
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